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Topics covered included applications of the SWMM to urban and rural catchments, the selection of design rainfall events for model use, a description of the Montreal Urban Community's wastewater interception program, several applications of other models in facility planning and design, development of a mini-computer version of the ILLUDAS model, and development of an urban/rural subcatchment hydrologic simulation model (SUBHYD).

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24-25 MAY 1979

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Office of Research and Development (RD-682)
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FOREWORD

A major function of the Research and Development program of the Environmental Protection Agency is to effectively and expeditiously transfer, to the user community, technology developed by those programs. A corollary function is to provide for the continuing exchange of information and ideas between EPA and users, and between the users themselves. The Stormwater Management Model (SWMM) users group, sponsored jointly with Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum.

This report, a compendium of papers presented at the last Users Group meeting, is published in the interest of disseminating to a wide audience the work of Group members.

Thomas A. Murphy
Deputy Assistant Administrator
Office of Air, Land and Water Use

ABSTRACT

This report includes fourteen papers, on various model-related topics. Presented at the semi-annual joint U.S.-Canadian Stormwater Management Model (SWMM) Users Group Meeting, held May 24-25, 1979, in Montreal, Quebec, Canada.

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DETERMINATION OF BASIN CHARACTERISTICS FOR AN URBAN DISTRIBUTED ROUTING RAINFALL-RUNOFF MODEL

By William M. Alley and Jack E. Veenhuis
U.S. Geological Survey

Presented at the
Storm Water Management Model Users
Group Meeting

Montreal, Canada

May 24 - 25, 1979

DETERMINATION OF BASIN CHARACTERISTICS FOR AN URBAN DISTRIBUTED ROUTING RAINFALL-RUNOFF MODEL

BY William M. Alley[1] and Jack E. Veenhuis[2]

INTRODUCTION

During the past decade, there has been a proliferation of urban rainfall-runoff models. Although many publications describe the mathematical formulation of these models, guidelines for determination of the basin characteristics to be used as part of the model input are almost nonexistent. Often, a listing of input and output data for a sample computer run is the extent of such guidelines.

The purpose of this paper is to discuss some of the procedures involved in determining basin characteristics for rainfall-runoff modeling. This discussion will center on the rainfall-runoff model documented by Dawdy and others (1978). However, transferability of much of the methodology to other rainfall-runoff models such as the Storm Water Management Model (Huber and others, 1975) should be possible. The approach described is not meant to provide hard and fast rules, but rather to serve as a guide. Different study objectives, study constraints, and models will require deviations from this guide.

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DRAINAGE BOUNDARIES

A basin characteristic which is of interest in virtually all rainfall-runoff studies is the total drainage area of the watershed. Determination of this parameter to the uninitiated may appear trivial--simply outline the basin boundary on a topographic map and determine the drainage area with a planimeter. However, considerable errors might result from using such a technique. This is illustrated in figure 1 which shows actual basin boundaries as determined after field inspection and the basin boundaries determined solely from U.S. Geological Survey 7-1/2 minute topographic maps. The ratios of drainage area determined solely from a topographic map to drainage area determined from field-verified boundaries for the 12 Denver basins ranged from 0.13 to 4.90. Major factors involved in these discrepancies include the effects of the street and storm-sewer systems, irrigation ditches, and the resolution capability of the maps.

All of the drainage basins shown in figure 1 have well-defined boundaries. However, this is not the case for all basins. Our experience has indicated that (1) well-defined urban watersheds with a hydraulically suitable site to measure discharge and without internal basin complications are difficult to locate, and (2) basin complications can be overlooked unless a thorough investigation of basin characteristics is undertaken. For example, the drainage area of an urban drainage basin may vary between different storms as a result of an irrigation ditch overflowing into the basin or exceedance of the capacity of an inlet or storm sewer resulting in conveyance of an unknown quantity of runoff to another basin. Other problems may occur such as a large storm sewer which discharges into two smaller storm sewers, one of which conveys an unknown quantity of water out of the basin. A basin,

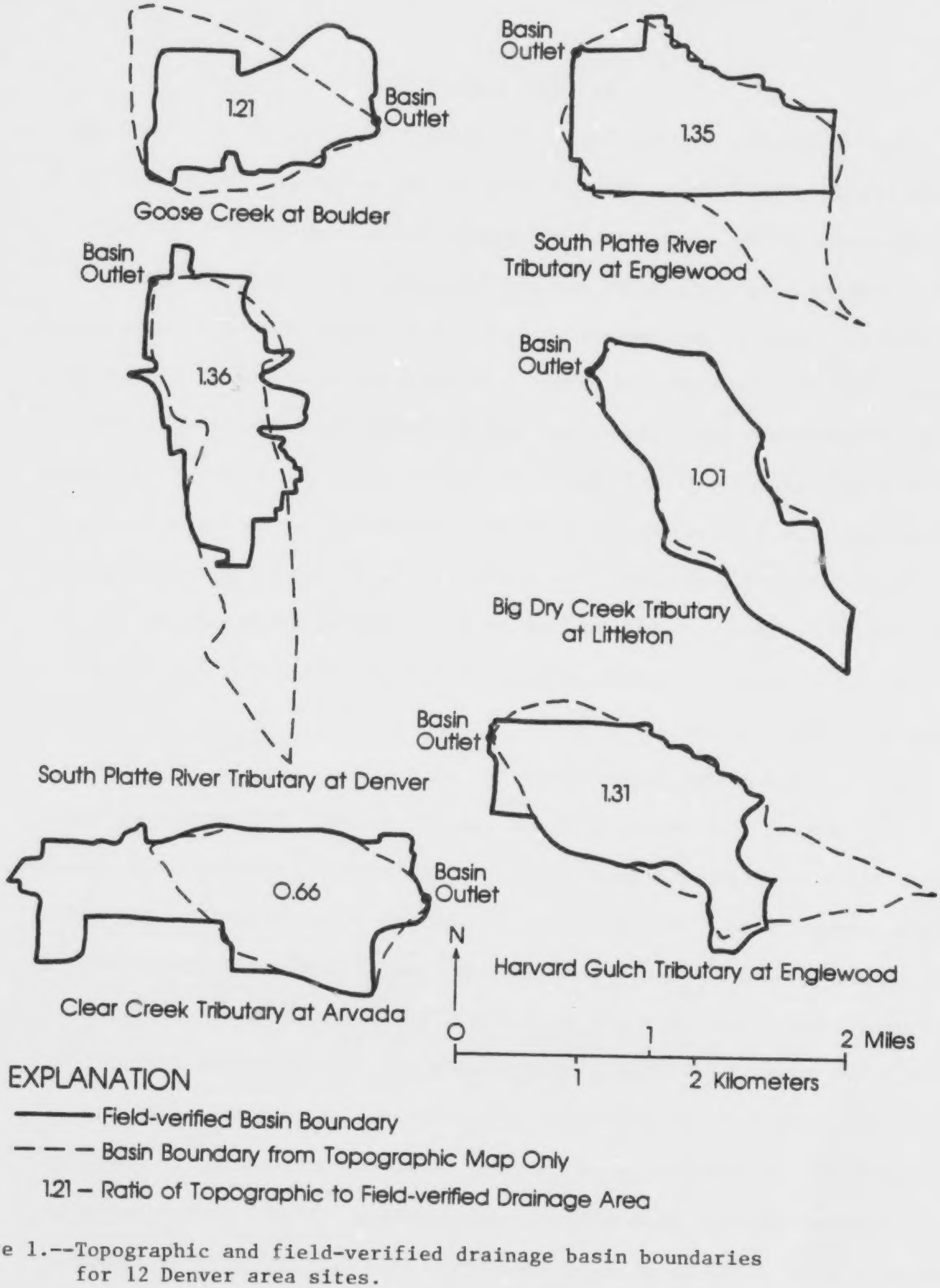

1.21
Basin Outlet
Goose Creek at Boulder
Basin Outlet
1.35
South Platte River Tributary at Englewood
Basin Outlet
1.36
South Platte River Tributary at Denver
Basin Outlet
1.01
Big Dry Creek Tributary at Littleton
Basin Outlet
1.31
Harvard Gulch Tributary at Englewood
0.66
Basin Outlet
Clear Creek Tributary at Arvada
N
0
1
2 Miles
1
2 Kilometers
EXPLANATION
Field-verified Basin Boundary
Basin Boundary from Topographic Map Only
1.21 – Ratio of Topographic to Field-verified Drainage Area

Figure 1.--Topographic and field-verified drainage basin boundaries for 12 Denver area sites.

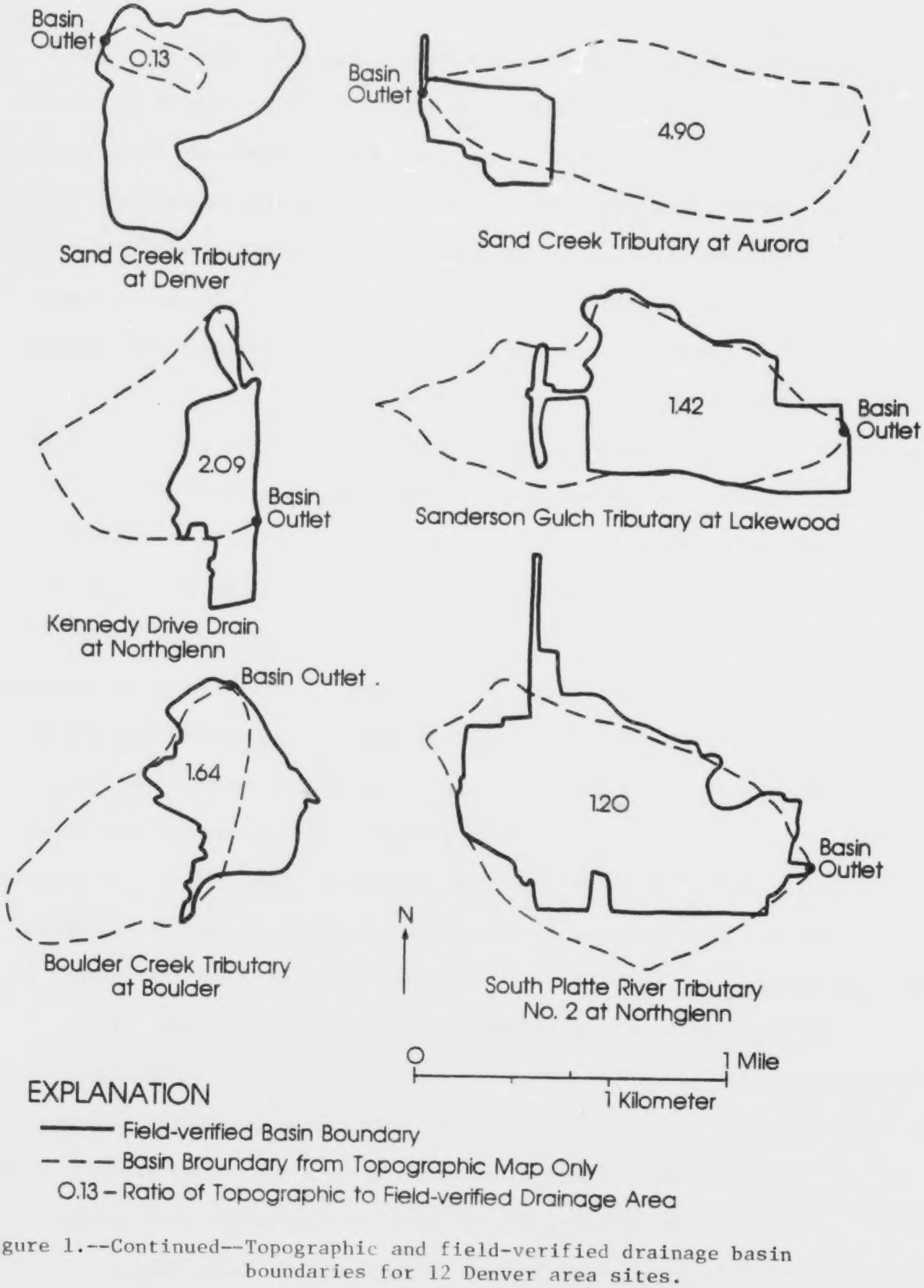

Basin Outlet
0.13
Sand Creek Tributary at Denver
Basin Outlet
4.90
Sand Creek Tributary at Aurora
2.09
Basin Outlet
Kennedy Drive Drain at Northglenn
1.42
Basin Outlet
Sanderson Gulch Tributary at Lakewood
Basin Outlet .
1.64
Boulder Creek Tributary at Boulder
1.20
Basin Outlet
N
South Platte River Tributary No. 2 at Northglenn
0
1 Mile
1 Kilometer
EXPLANATION
Field-verified Basin Boundary
Basin Broundary from Topographic Map Only
0.13 – Ratio of Topographic to Field-verified Drainage Area

Figure 1.--Continued--Topographic and field-verified drainage basin boundaries for 12 Denver area sites.

particularly in newer developments, may have a complex, unregulated detention reservoir. All of these problems and others have been identified in actual field studies of potential sites. Areas with undersized storm sewers, storm-sewer inlets, or irrigation ditches are particularly susceptible to problems. Field verification during rainfall and snowmelt runoff is sometimes necessary to resolve questions about basin response to runoff. Also, consultation with local government officials familiar with urban drainage, zoning, and/or public works can be invaluable.

These observations indicate that basin characteristics are not only important for rainfall-runoff modeling. In addition, if a basin is to be instrumented for a rainfall-runoff data-collection program, a thorough investigation should be conducted at the outset of the program to insure the usefulness of the basin. Detailed determination of segment characteristics for modeling would not be necessary at that time. However, prior to initiating an expensive data-collection program, it is important to assure that there are no basin complications which can negate the utility of that basin for inclusion in the study and to determine general basin characteristics which can be used to assist in final selection of basins. Occasionally, an adequate description of the storm-sewer system draining a basin may not be available. This factor alone may limit the utility of the basin for data collection.

It is important to recognize that urban drainage basins are often "watersheds in transition" and that the basin characteristics may change significantly from year to year. Some of these changes, such as in impervious area, can be monitored from aerial photographs. However, other changes, such as in the sewer system, often cannot be detected from aerial photographs. There have been instances when renovations to the storm-sewer system were made during a rainfall-runoff data-collection program. Upon analysis of the data, the old storm-sewer maps had been lost or destroyed and the characteristics of the previous storm-sewer system no longer known.

Usually, basins selected for data collection are not expected to undergo significant changes. However, this is often difficult to predict. Occasionally, a basin might be selected with the intent of monitoring changes in runoff characteristics as the basin develops. In either case, an annual update should be made of the characteristics of all basins included in a rainfall-runoff data-collection program.

BASIN SEGMENTATION

The objective of basin segmentation is to characterize a basin in terms of a discrete set of segments which account for the essential basin properties. Each segment is assumed to be homogeneous in its properties. The model developed by Dawdy and others (1978) considers four types of segments:

1. Overland-flow segments
2. Channel segments
3. Reservoir segments
4. Junction segments

Overland-flow segments receive uniformly distributed lateral inflow from excess precipitation. They represent a rectangular plane of a given overland-flow length, roughness, slope, and percent imperviousness.

Channel segments receive upstream inflow from as many as three other segments, including combinations of other channel segments, reservoir segments, and junction segments. They also receive lateral inflow from overland-flow segments.

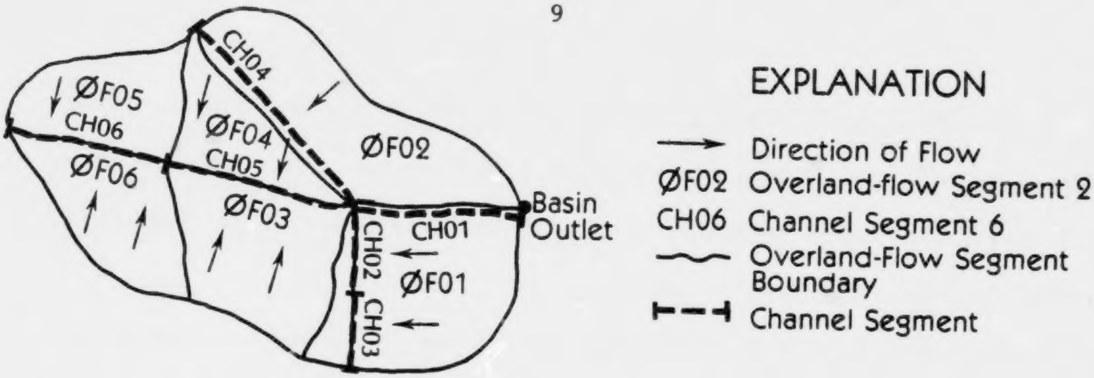
Reservoir segments provide for either linear-storage routing or modified-Puls routing (Soil Conservation Service, 1972). They can be used to describe an on-channel detention reservoir. Alternately, they can be used to simulate culverts which detain water due to limited capacity and for which outflows are uniquely described as a single-valued function of storage behind the culvert.

Junction segments are used when more than three segments contribute inflow to the upstream end of a segment or as input-hydrograph points where the user may specify an input hydrograph for each storm.

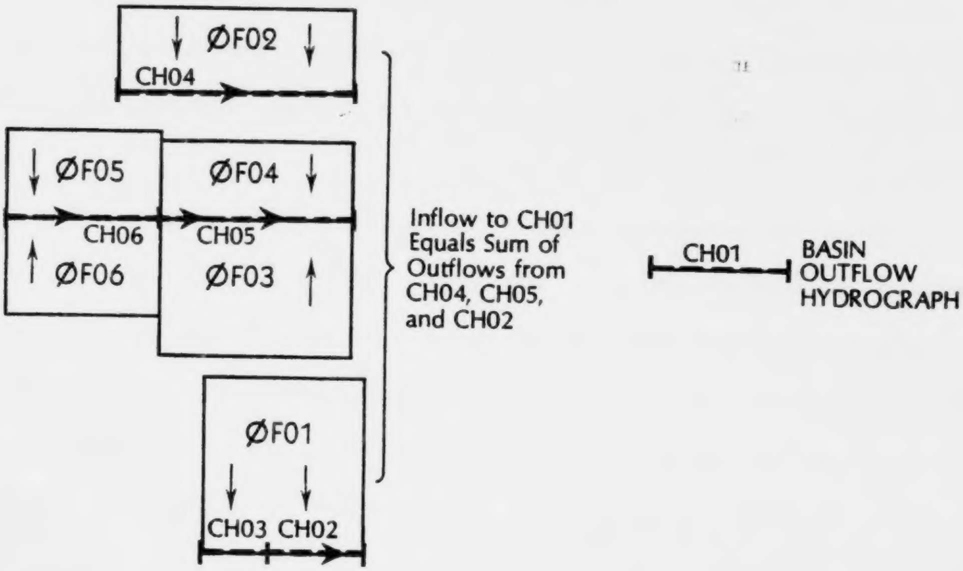
Most of the discussion in this paper centers on overland-flow and channel segments. The interrelationships of rainfall, overland-flow segments, and channel segments are illustrated in figure 2 which shows the discretization (segmentation) of a simple urban watershed.

Consider the discretization of an urban drainage basin into a set of model segments; the following steps might be followed:

1. Obtain available information on the basin including storm-sewer maps, topographic maps, and land-use maps. Obtain 1-inch equals 100- to 1,000-foot aerial photography. Aerial photographs of this scale should be available either from local governments or from a local engineering firm. If no aerial photographs are available, then a contract for obtaining them might be considered. These services can usually be obtained at $200 to $1,500 per basin, depending on the number of basins flown, the size of the basins, and the scale of the photographs, among other factors. Mylar prints should be obtained so that work copies can be made of each photo.


ØF05
CH06
ØF06
CH04
ØF04
CH05
ØF02
ØF03
Basin Outlet
CH02
CH01
ØF01
CH03
EXPLANATION
Direction of Flow
ØF02 Overland-flow Segment 2
CH06 Channel Segment 6
Overland-Flow Segment Boundary
Channel Segment

(a) PLAN VIEW OF DRAINAGE BASIN


ØF02
CH04
ØF05
ØF04
CH06
CH05
ØF06
ØF03
ØF01
CH03 CH02
Inflow to CH01 Equals Sum of Outflows from CH04, CH05, and CH02
CH01
BASIN OUTFLOW HYDROGRAPH

(b) SCHEMATIC REPRESENTATIONS OF MODEL SEGMENTS

Segment	Inflow to Segment	
	Lateral Inflow	Upstream Inflow
ØF01	Rainfall Excess	—
ØF02	" "	—
ØF03	" "	—
ØF04	" "	—
ØF05	" "	—
CH01	—	CH02, CH04, CH05
CH02	ØF01	CH03
CH03	ØF01	—
CH04	ØF02	—
CH05	ØF03, ØF04	CH06
CH06	ØF05, ØF06	—

(c) SEGMENT INTERRELATIONSHIPS

Figure 2.--Discretization of an urban basin into segments.

2. Assuming aerial photographs are obtained, mark the location of the drainage network and inlets on a set of the work photos. This will require some field verification to assure that storm-sewer maps and other information obtained for the basin are "as built".

3. Mark overland- and street-flow directions on the aerial photographs, particularly at street intersections. This step will require some field inspection of the basin. Many times, flow directions on streets will be difficult to determine. Use of hand levels or field verification during periods of rainfall or snowmelt might be necessary to resolve questions about flow directions.

4. Using the marked-up photos, the basin can then be segmented.

There are no exact rules on how to proceed and segmentation will be a function of the model used and the purpose of the study. However, as an example, some general rules of thumb for basin segmentation for application of the model documented by Dawdy and others (1978) are as follows:

A. It is often easiest to begin segmentation by starting at the downsream end of a basin.

B. Channel segmentation and overland-flow segmentation are generally done at the same time as the two are highly interrelated.

C. The more highly developed part of the basin generally will require a more detailed breakdown into segments because of additional complexity of the drainage and the greater contribution to watershed runoff per unit of land area.

D. This model, as do most urban-runoff models, assumes all overland-flow segments are rectangular. Therefore, attempts should be made to create overland-flow segments which approach this shape.

E. Attempts should be made to obtain segments of fairly uniform characteristics throughout the segment since this is assumed by the model.

F. Channel intersections often exert a controlling influence on overland-flow segmentation because each overland-flow segment must drain to a channel segment.

G. Channel segmentation is also governed to a certain extent by the overland-flow segmentation; although, this effect has been minimized. The lateral inflow rate to the channel segment that picks up the overland flow is expressed in terms of overland outflow in cubic feet per second per foot of channel length. It suffices to model the flow from a 1-foot wide strip of overland flow having a characteristic length, slope, percent imperviousness, and roughness coefficient. Therefore, an overland-flow segment can be specified as contributing lateral inflow to a number of different channel segments. For example, overland-flow segment 0F01 drains to channel segments CH02 and CH03 in figure 2. The model routes the flow through segment 0F01 only once and uses outflow from 0F01 (in cubic feet per second per foot of channel length) as lateral inflow to both channel segments.

H. Segmentation into the fewest number of segments which will preserve the essential basin hydrologic-response characteristics is desired. Finding a suitable simplified segmentation is important for derivation of runoff frequency curves which can be expensive to simulate. One approach might be to perform various levels of segmentation and to determine the sensitivity of the model to these different levels. Leclerc and Schaake (1973) have demonstrated that when a detailed segmentation is simplified, the changes most likely to be observed are a faster response of the rising limb and recession limb of the simulated hydrograph. Experience with the model has shown that the "optimum" number of segments may depend more on basin complexity and subbasin hydrograph interest than basin size.

I. Break points between channel segments should occur at channel intersections (for example, the intersection of CH01, CH02, CH04, and CH05 in figure 2), at points of considerable change in channel characteristics, at points where sewer surcharge or culvert surcharging may restrict the flow, and at points of interest for subbasin hydrographs.

J. Modified-Puls reservoir segments can be used to simulate culverts which detain water due to limited capacity and for which outflows are uniquely described as a single-valued function of storage behind the culvert. A culvert rating and stage-storage relationship will be needed to determine the outflow-storage relationship required by the modified-Puls formulation.

SEGMENT CHARACTERISTICS

Some of the watershed characteristics are specified as pertaining to the entire drainage basin. For example, in general application a single set of infiltration and soil-moisture parameters are specified for the entire basin. The model includes an option for calibrating these parameters using measured rainfall and runoff data and a modified-Rosenbrock optimization technique (Rosenbrock, 1960; Dawdy and O'Donnell, 1965). Other watershed characteristics are required for each segment.

Overland-Flow Segments

Characteristics required for each overland-flow segment include percent impervious area, length of overland flow, overland-flow slope, and a roughness coefficient.

Percent Impervious Area

Two types of impervious surfaces are considered by the model. The first type, effective impervious surfaces, are those impervious areas that are directly connected to the channel drainage system, such as streets and roofs that drain onto driveways and streets. The second type, noneffective impervious surfaces, are those impervious areas that drain to pervious areas. An example of a noneffective impervious area is a roof that drains onto a lawn.

Rain falling on noneffective impervious areas is assumed to run off onto the surrounding pervious area. The model assumes that this occurs uniformly over the entire pervious area. This volume of water is added to the rain falling on the pervious areas prior to computation of infiltration.

Several studies (Graham and others, 1974; Proctor and Redfern Limited and James F. MacLaren Limited, 1976) of a similar model, the Storm Water Management Model II (Huber and others, 1975), have shown simulated runoff volumes and peak flows to be highly sensitive to the percent effective impervious area. A number of studies have outlined methods to determine impervious area; however, few studies have separated effective from noneffective impervious area. Most studies of this type have related effective impervious area to the minimum ratio of runoff/precipitation measured for small storms (Crawford, 1971; and Miller, 1978). The rationale for this approach is that for basins with highly permeable soils, runoff from small storms originates almost entirely from the effective impervious area in the watershed. Limitations of this approach are the large scatter often observed in runoff/precipitation plots, the requirement for rainfall-runoff data from the watershed, the methodology may not apply to basins with soils of moderate to low permeability, and the sensitivity of the method to errors in precipitation and flow measurements.

An alternate approach used in Denver rainfall-runoff studies was as follows. The lengths and widths of all streets in each overland-flow segment were measured from aerial photos and the total area of streets for each overland-flow segment was determined. All roofs, parking lots, and other impervious areas which could not be identified as either effective or non-effective were then field inspected and all of these areas which were effectively impervious were colored in red on the aerial photo. If only part of a roof or other impervious area was effective, then only that part which was effective was colored. The areas of several representative roofs in each overland-flow segment were then measured from the aerial photographs and the average of these areas multiplied by the number of effective impervious roofs for each overland-flow segment. A similar approach was used for driveways. Finally, the area of remaining effective impervious areas, such as parking lots, was planimetered. The combined field and office work required for this approach was approximately 4-person days per square mile of basin for a highly developed watershed.

The above methodology was assumed to be consistent with the sensitivity of runoff simulations to estimates of percent effective impervious area as well as the inherent subjectivity of the analysis. For example, it was often difficult to determine which part of a house roof was effective impervious area, particularly for houses with a downspout close but not connected to a driveway.

Other approaches might include a random sampling of roofs or a grid overlay approach. Important requirements appear to be a large-scale aerial photo to delineate impervious areas and some field inspection to differentiate effective and noneffective impervious areas. Results from Denver studies suggest that there may be some transferability of impervious area information throughout a metropolitan area for selected land-use types.

Table 1 summarizes the analyses of impervious areas for 12 urban basins in the Denver area, including 91 subbasins (overland-flow segments). As expected, for single-family residential development the percentage of area that is impervious appears to decrease as the lot size increases. Despite this trend, the percentage of impervious area that is effective appears to be relatively constant for all single-family residential lot sizes. Remarkably, the five subbasins with trailer courts had essentially the same mean and range of values for percentage of impervious area that is effective as did the single-family residential subbasins.

Interesting aspects of the data summarized in table 1 are that five different people were involved in their collection and that single-family residential developments of many different ages and parts of the Denver metropolitan area were sampled. The data suggest that for metropolitan Denver, in the absence of field data for basins or parts of basins in single-family residential land use (including trailer courts), it might be assumed

Table 1.--Total- and effective-impervious areas for urban basins in metropolitan Denver

Land use	Lot size (acres)	No. of subbasins sampled	Percentage of area that is impervious		Percentage of impervious area that is effective	
			Mean	Range	Mean	Range
Single-family residential	$\frac{1}{6} - \frac{1}{5}$	59	41	37-46	58	54-63
	$\frac{1}{5} - \frac{1}{4}$	3	42	38-46	56	52-61
	$\frac{1}{4} - \frac{1}{2}$	3	26	24-28	59	55-63
	$\frac{1}{2} - 1$	3	15	12-18	55	52-59
Trailer Courts	-	5	48	40-52	57	53-64
Multifamily residential	-	4	72	61-86	75	71-79
Commercial	-	3	93	92-94	92	80-99
Industrial	-	11	1/	37-78	1/	37-100

1/ Mean not reported because of large variations that approximately 57 percent of the total impervious area is effective. The total impervious area might be measured or the mean percentages shown in table 1 could be used. Only subbasin sampling in other urban areas can confirm the values shown in table 1 for use outside the Denver area.

The seven multifamily residential and commercial subbasins that were sampled had values for the percentage of impervious area that is effective ranging from 71 to 99 percent. Additional subbasin sampling might be warranted before using these as representative values. Industrial subbasins showed large variations in both percent impervious area and the percentage of impervious area that is effective. These variations result from the large building sizes and the wide variations of storm-water-management practices, ranging from connecting directly to storm-sewer lines to draining onto vacant land.

Length of Overland Flow

The length of overland flow, in feet, can be computed as the area, in square feet, of the overland-flow segment divided by the length, in feet, of the channel(s) into which it contributes lateral inflow. Leclerc and Schaake (1973), and Huber and others (1975) discuss other methods of determining length of overland flow; however, these options are not included in the present version of this model.

Overland-Flow Slope

Unless a large scale, smaller contour interval, topographic map of the basin is available, the expense of contracting services for such a map is generally not warranted. Overland-flow slopes can be estimated from U.S. Geological Survey 7 1/2-minute topographic maps. One method would be to determine a weighted average slope from representative cross sections of the overland-flow segment using the following equation:

$$\text{Slope} = \frac{\sum_{i=1}^{n} S_i \; L_i}{\sum_{i=1}^{n} L_i} \qquad (1)$$

where

S_i = the slope of the ith cross section,

L_i = the length of the ith cross-sectional line, and

n = the number of sampling lines.

An alternate would be to use the following equation described by Wisler and Brater (1959):

$$\text{Slope} = \frac{DL_c}{A} \qquad (2)$$

where

D = contour interval, in feet,

L_c= total length of contours for segment, in feet, and

A = area of segment, in square feet.

Roughness Coefficients

Roughness coefficients used by the model are Manning's "n" values. Because of the shallow flows involved, roughness coefficients are not static but rather vary with the depth of flow and with variations in the properties of the watershed such as grain size, slope, and density of grass blades. An additional complication results from overland-flow segments having pervious and impervious areas dispersed throughout the segment. An alternative provided by the model is to separate the runoff from an overland-flow segment into a pervious and impervious portion and to route each separately with different roughness coefficients. Successful simulations have been made assuming 0.014-0.016 for the impervious-area roughness coefficient and 0.15-0.50 for the pervious-area roughness coefficient. Some guidance on roughness coefficients for pervious areas is given by Chen (1976).

Channel Segments

Segment characteristics required for each channel segment include channel length, channel slope, Manning's "n", and cross-section geometry.

Channel Length

Channel length is a relatively easy parameter to measure from topographic maps, storm-sewer maps, or aerial photos. Lengths of storm sewers are often marked on storm-sewer maps.

Channel Slope

Channel slope is often measured as the difference in elevation at points 10 percent and 85 percent of the distance along the channel, measured from the downstream end of the channel, divided by the distance between the two points. Slopes of storm sewers are often marked on the storm-sewer maps.

Manning's "n"

Chow (1959) presents a good discussion of Manning's "n" values for channels. Typical values might range from 0.02 to 0.12.

Cross-Section Geometry

The model uses kinematic wave formulation whereby a relationship is specified between the area of flow (A) and the rate of flow (Q) as

$$Q = \alpha A^m$$

The model parameters α and m can be determined by analysis of the cross-section geometry. For example, if for a given channel segment, the wetted perimeter (P) can be expressed as a power function of the area of flow by:

$$P = aA^{b} \tag{4}$$

where a and b are constants, then Manning's equation for the segment can be expressed as

$$Q = \frac{1.49\sqrt{S}}{n\ a^{2/3}}\ A^{(5/3\ -\ 2/3\ b)} \tag{5}$$

Therefore,
$$\alpha = \frac{1.49\sqrt{S}}{n\ a^{2/3}} \tag{6}$$

and
$$m = \frac{5}{3} - \frac{2}{3}\,b \tag{7}$$

where S is the segment slope and n is Manning's "n".

The kinematic-wave parameters α and m are computed internally by the model for rectangular conduits, circular pipes, and triangular cross sections based on geometric properties of these cross sections. They may also be specified externally by the user either based on flow measurements or from equations 6 and 7.

In natural channels, the question arises as to whether a representative channel cross section should be surveyed and equations 4, 6, and 7 used to determine α and m or whether a simple rectangular, circular, or triangular geometry can be assumed. A sensitivity analysis of rainfall-runoff simulations was performed for South Utoy Creek Tributary at Woodberry Avenue at East Point, Georgia, (the Woodberry basin) in order to investigate this subject.

The Woodberry basin is 0.21 square miles in area with drainage by a natural channel down the center of the basin. A schematic of this basin is shown in figure 3. Basin characteristics for rainfall-runoff modeling were available (E. J. Inman, written commun., 1979). In addition, representative cross sections of each of the channel segments had been surveyed from which the values of a and b in equation 4 could be determined (E. J. Inman, written comm., 1979). Alpha and m for each channel segment were then computed using equations 6 and 7. Three computer runs were made for a set of 19 measured rainfall-runoff periods. The first computer run was made using values for α and m as computed using equations 6 and 7. The second computer run was made using values of α and m as computed using "rough" approximations of channel cross sections as having rectangular geometry. The third computer run was made using "best" approximations of channel cross sections as having either rectangular or triangular geometry.

The channel geometries used in the three computer runs are shown in figure 4. Note that the same cross sections were used for channel segments CH20 and CH21 for both the "rough" and "best" approximation runs. The results of the three simulations are shown in table 2. Assuming that simulated peaks for the first computer run are the "true" values, then the average errors were 5.3 cubic feet per second for the "rough" approximation run and 1.0 cubic foot per second for the "best" approximation run. Standard error of estimate was 17 percent for the "rough" approximation run as compared to 1.9 percent for the "best" approximation run.

The results of this sensitivity analysis indicate that, in general, determination of values for a and b in equation 4 is not necessary and that a "best fit" rectangular, circular, or triangular geometry can be assumed

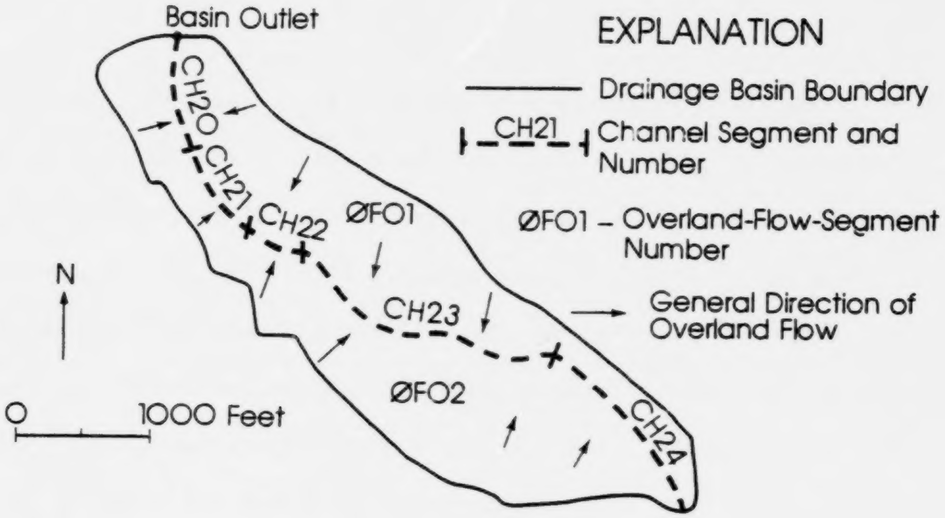

Basin Outlet
EXPLANATION
Drainage Basin Boundary
CH21 Channel Segment and Number
ØFO1 – Overland-Flow-Segment Number
General Direction of Overland Flow
N
0 1000 Feet
CH20
CH21
CH22
CH23
CH24
ØFO1
ØFO2

Figure 3.--Schematic of Woodberry basin showing segmentation for rainfall-runoff modeling.

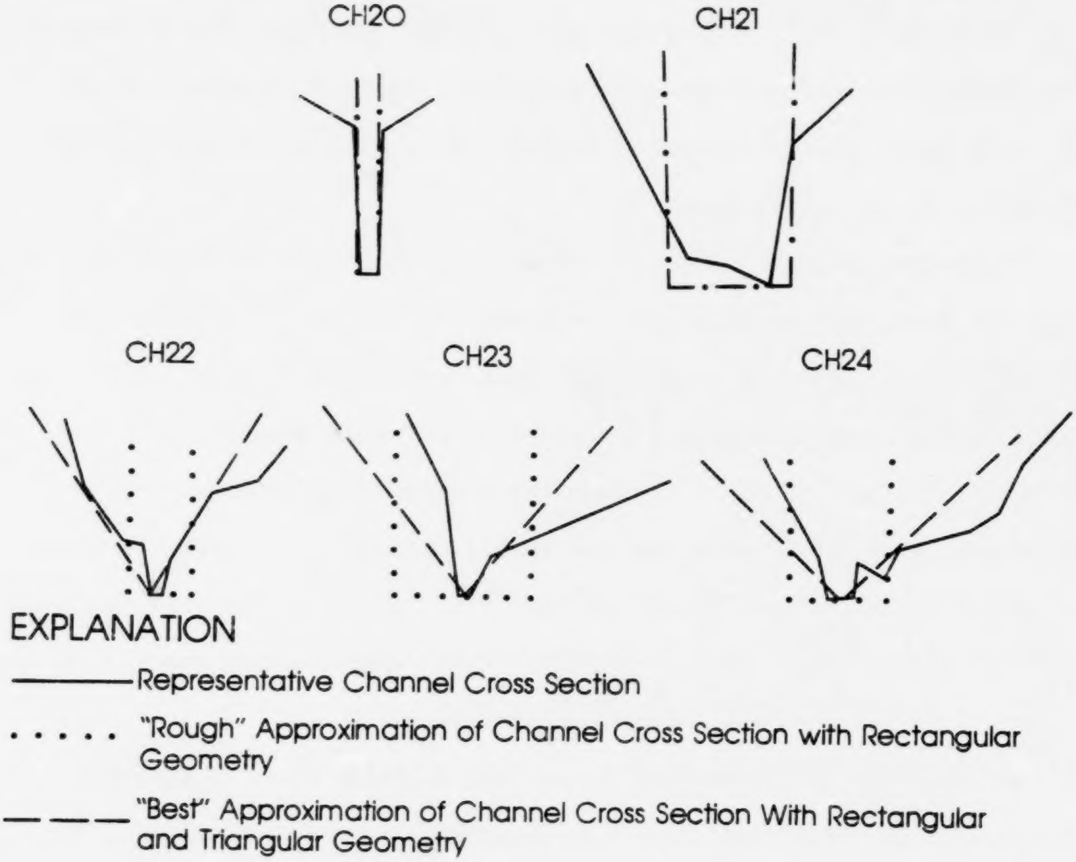

CH20
CH21
CH22
CH23
CH24
EXPLANATION
Representative Channel Cross Section
"Rough" Approximation of Channel Cross Section with Rectangular Geometry
"Best" Approximation of Channel Cross Section With Rectangular and Triangular Geometry

Figure 4.--Actual and approximated channel geometry.

Table 2.--Simulated peak flows for various channel-geometry assumptions

Runoff Period	Simulated peak flow, in cubic feet per second		
	Run No. 1 [1/]	Run No. 2 [2/]	Run No. 3 [3/]
1	105	100	105
2	37.5	32.9	37.5
3	10.0	8.4	10.2
4	32.7	27.7	33.2
5	139	137	144
6	59.0	52.7	59.4
7	29.2	20.0	30.0
8	21.7	15.1	22.1
9	70.5	65.8	72.1
10	37.1	30.7	37.3
11	104	99.6	107.7
12	52.2	51.0	52.5
13	24.8	23.0	24.9
14	102	99.9	103
15	36.2	28.0	36.7
16	96.4	84.0	95.2
17	115	103	115
18	104	101	104
19	110	105	114

[1/] Alpha and m values for channel segments based on representative channel cross sections and equations 4, 6, and 7

[2/] Alpha and m values for channel segments based on "rough" approximation with rectangular geometry

[3/] Alpha and m values for channel segments based on "best" approximation with rectangular and triangular geometry for each channel segment. However, care should be exercised in determining these "best fit" channel geometries and dimensions. An approach used in Denver studies was to estimate a simple channel geometry in the field without resorting to detailed surveying of channel cross sections.

The capacity of rectangular-conduit and circular-pipe segments is limited in the model to nonpressurized-flow capacity. If that capacity is exceeded during a storm, provision is made to store the excess water arriving at the upstream end of the segment. The volume stored increases without upper limit as long as the upstream inflow exceeds segment capacity. After the upstream inflow drops below segment capacity, the volume stored is released to the segment. The upstream inflow to the segment remains at the maximum capacity until the water stored at the upper end of that segment has been released. In the real world, the capacity of a sewer may be controlled by inlets which restrict flow in the sewer to less than full-pipe flow. A second possibility is that a sewer may flow under pressure, thus having more capacity than predicted using kinematic-wave theory. A third possibility is that once a sewer is flowing full, additional inflow to the sewer is transferred to streets parallel to the sewer system. These situations can, at times, be approximated by adjusting the size of the circular or rectangular segment appropriately. A modified-Puls reservoir segment can be used to simulate culverts which detain water due to limited capacity and for which outflows are uniquely described as a single-valued function of storage behind the culvert. More complex situations may call for revision of the model or use of a dynamic-wave model such as that presented by Water Resources Engineers, Inc., (1972).

SUMMARY AND CONCLUSIONS

This paper has described some of the difficulties, procedures, and limitations of determining basin characteristics for rainfall-runoff modeling. A procedure for discretization of an urban drainage basin into a set of model segments has been described including 10 general rules of thumb for basin segmentation and approaches for determining segment characteristics. The discussion has centered on a rainfall-runoff model presented by Dawdy and others (1978). However, transferability of much of the methodology to other rainfall-runoff models such as the Storm Water Management Model (Huber and others, 1975) should be possible.

During investigations to identify urban watersheds for a rainfall-runoff data-collection program, it was noted that well-defined urban watersheds with a hydraulically suitable site to measure discharge and without internal basin complications are difficult to locate. Basin complications can be overlooked unless a thorough investigation of basin characteristics is undertaken prior to final site selection.

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SWMM USERS CONFERENCE

MONTREAL, MAY 24-25, 1979

EVALUATION OF PROPOSED URBAN RUNOFF CONTROL ALTERNATIVES

by

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ABSTRACT

This paper advocates using runoff frequency curves as the basis of decision in drainage systems planning and design.

A conceptual framework for deriving runoff-related frequency curves from rainfall probability distributions is presented and illustrated for the case where the runoff statistic is a function of only two random variables.

Stochastic simulation is discussed as one solution procedure and examples of application are given.

Complete derivation of a runoff related frequency curve is probably too costly for pratical planning and design applications. The stochastic simulation procedure described suggests that the use of an ensemble of historical storms selected from observed records can be a good pratical trade-off between a design storm and complete derivation of frequency curves from rainfall.

1.0 INTRODUCTION

Storm and combined drainage networks are currently designed for a design rainfall event, a storm with a given duration and mean rainfall intensity, both interlinked through intensity-duration curves. The spatial distribution of the design storm is normally assumed uniform whereas its temporal distribution (the hyetograph) may be uniform, triangular or of the Chicago type. With intensity-duration curves available, and with an estimated time of concentration of the basin, at the upstream end of the element designed, the design rainfall exterior variables (intensity and duration) are known once the recurrence interval for the design has been selected.

The recurrence interval associated with the design rainfall event is one measure of the risk assumed, and more significantly, an explicit recognition of the randomness of the rainfall process. When the rational formula is chosen as transfer function, the recurrence interval of its runoff statistic (peak runoff) is equal to the recurrence interval of the storm; this follows from the linearity relationship between storm intensity and peak runoff.

Recent advances in urban drainage have been centered around many aspects of the drainage problem, namely: a) improvement or development of rainfall-runoff models with particular focus on urban runoff applications, b) assessment of urban runoff control alternatives such as temporary storage of urban runoff, c) assessment of water quality control facilities for combined sewers and d) description of the rainfall process at a scale relevant to urban drainage analysis.

Very little attention has been given to the overall question of how to assess a given drainage network, of how to compare two or more drainage alternatives (both network and control facilities). Indeed urban drainage studies even those using sophisticated rainfall-runoff models, are still embedded in the design storm framework.

While it is accepted that the recurrence interval of a given runoff statistic is not identical to the recurrence interval of the runoff producing rainfall event, few studies have been concerned with the problem of selecting the appropriate recurrence interval based on runoff (quantity or quality) statistic(s), or of comparing (and assessing) various control runoff alternatives. The San Francisco study moved in the right direction with continuous simulation of urban runoff from a record of observed storms; recurrence interval were then defined on runoff rather than rainfall statistics.

This paper presents a theoritical framework for deriving runoff frequency curves from rainfall probability distributions , discusses numerical methods of solution and assesses the faisability of deriving urban runoff frequency curves within a drainage project study.

2.0 LITTERATURE REVIEW

Perkins (1970) presented a procedure for computing frequency curves of flood stages in a flood plain.

Eagleson (1971, 1972) derived analytically the flood frequency curve of some natural catchments in Connecticut from the rainfall probability distribution of the average excess intensity and duration; he found that the analytic expression for the cumulative distribution of peak runoff reproduces well the essential features of the observed curves.

Fleming and Franz (1971) compared four methods for deriving flood frequency curves for small watersheds; in a test of eleven watersheds, the simulation approach proved superior to the other methods.

Leclerc & Schaake (1972) derived by stochastic simulation the flood frequency curve of a natural catchment; they concluded that the simulation approach is feasible and reliable and that future work should be devoted to improve the computational efficiency of the procedure developed.

In 1973, they used this approach to derive frequency curves of urban runoff statistics, and proposed the use of these curves for assessing the potential impact of urban development on urban runoff and for estimating the relative efficiency of proposed runoff control alternatives.

Howard (1976) proposed an analytical model of storage and treatment-plant overflows alternatives based on probability distributions of rainfall variables; this model develops statistics of untreated discharge through mathematically derived probability functions. This model can be a screening model for directing more detailed simulation analysis of the most interesting alternatives.

Bras and Chan (1978) derived an analytical expression of the volume of water above certain threshold discharge rates resulting from runoff. This expression is for an overland flow segment, whose runoff is estimated through the kinematic wave model, and for exponentially distributed storm duration and effective intensity.

Dendrou and Delleur (1978) argued for introducing the notion of risk at the planning stage of an urban storm drainage study. They proposed a complete set of reliability measures, associated with the three basic function of protection against local street-floods, contribution to downhill river floods and pollution from runoff.

Marsalek (1978) reported on the results of research on the implications in the use of the design storm concept in urban drainage design and planning. Several statistics were considered; based on his results, he suggested that design storms could be replaced by historical storms in future drainage design.

3.0 THEORETICAL FRAMEWORK

Deriving the frequency curve of a runoff statistic from the probability distribution of relevant rainfall variables is essentially an exercise in derived distributions; this topic is discussed among others by Benjamin and Cornell (1970). In this section a general theoretical framework is first presented, followed by a graphical illustration for the particular case where the runoff statistic is assumed to be a function of only two rainfall random variables.

3.1 GENERAL CASE

Let Q be a runoff statistic and $\underline{\theta}$ be a vector of variables such that

$$Q = Q(\underline{\theta}) = Q(\underline{\theta_1}, \underline{\theta_2}) \tag{1}$$

The mathematical relationship $Q(\underline{\theta})$ between Q and θ is equivalent to a mathematical model of the rainfall-runoff process. $\underline{\theta}$ is made of the parameters of the model and of the input variables which are assumed random variables, $\underline{\theta_2}$. Then the derived distribution of Q is obtained through

$$F_Q(q) = \int\int\int\ldots\int_{R(q)} f_{\underline{\theta_2}}(\underline{\theta_2})\, d\underline{\theta_2} \tag{2}$$

where $f_{\underline{\theta_2}}(\underline{\theta_2})$ is the joint probability function of $\underline{\theta_2}$.

$F_Q(q)$ is the cumulative density function of Q, and R(q) is the region where the vector $\underline{\theta}$ leads to values of Q that are less than or equal to q. The task then consists mainly in establishing the boundaries of the region R(q), once $f_{\underline{\theta_2}}(\underline{\theta_2})$ is known.

The rainfall-runoff model is considered here a deterministic transformation function, the random components being accounted for in the inputs to the model.

$F_Q(q)$ is the derived probability distribution of all rainfall-runoff events. To obtain the frequency curve, say on an annual basis, additional effort is needed to identify the maximum Q that occured in a given year, or the ensemble of the r largest Q's. The calendar time must then be introduced to obtain an estimate of the recurrence interval.

3.2 CASE OF TWO RANDOM VARIABLES

Equation 2 is a double integral when $\underline{\theta}$ contains only two random variables; in this case a graphical illustration of the derivation mechanisms can be made.

Let t_{r_e} be the storm duration and $\bar{i}_e$ be rainfall excess mean intensity. Their joint probability density function, $f(\underline{\theta_2}) = f(\bar{i}_e, t_{r_e})$ is known and is shown in figure 1 for the double exponential case.

The integration region $R(q)$ is defined by the three planes $(\bar{i}_e, t_{r_e})$, $(f(\bar{i}_e, t_{r_e}), \bar{i}_e)$ and $(f(\bar{i}_e, t_{r_e}), t_{r_e})$, the surface $f(\bar{i}_e, t_{r_e})$ and the vertical plane through the contour line $Q = q$. In figure 1, $Q = 200$ ft^3/s. The volume in this region is the probability that Q is smaller than or equal to 200 ft^3/s, that is $F_Q(200)$. $G_Q(200)$ is the volume taken-off $(G_Q(q) = 1 - F_Q(q))$.

To obtain the frequency curve, the following equation, derived by Eagleson (1972) is used:

$$\frac{1}{T_E} = n\, G_Q(q)$$

where n is the mean number of runoff producing rainfall events in a year and T_E is the recurrence interval on an annual exceedance basis (years).

The role of the transformation function, thus of the rainfall-runoff model is emphasized: the model defines the region of integration $R_Q(q)$ as it locates the contour where Q is equal to q. All

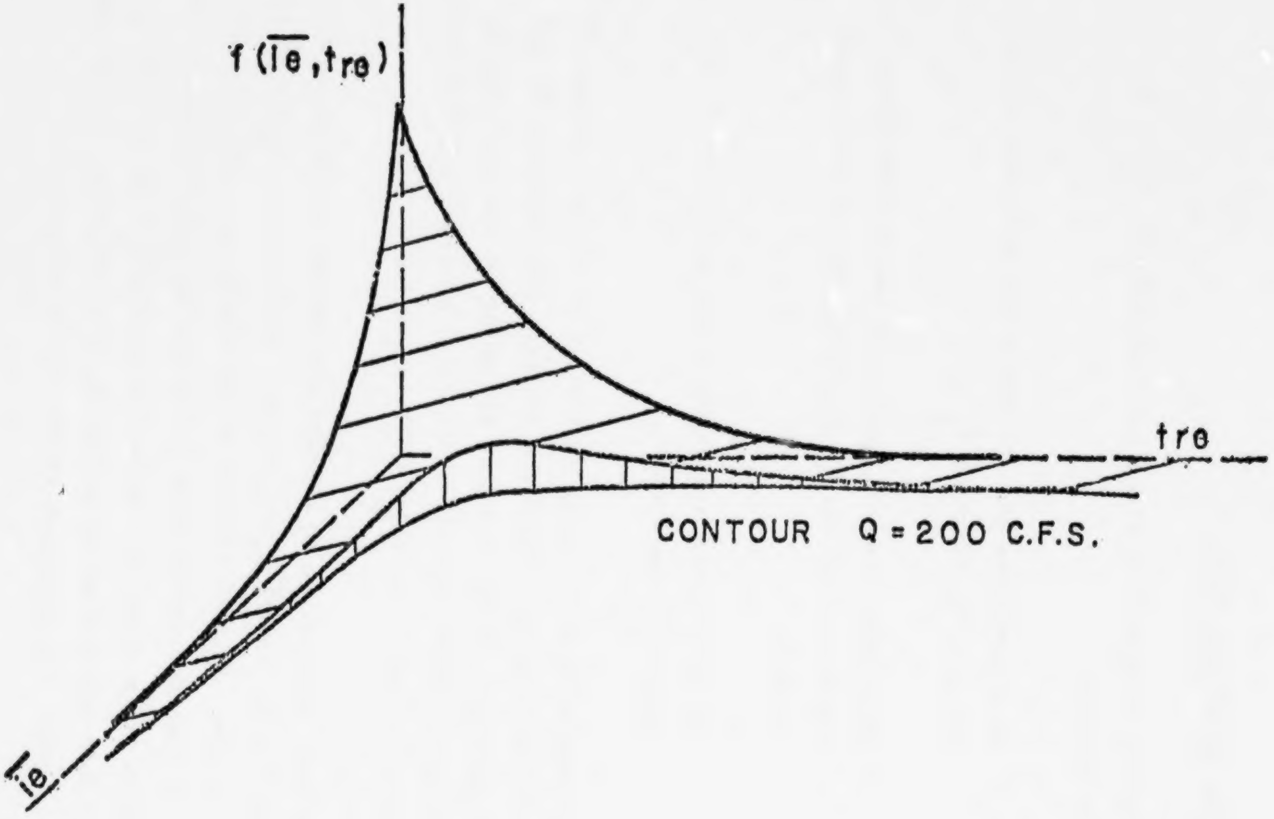
$f(\overline{Ie}, t_{re})$
t_{re}
CONTOUR Q = 200 C.F.S.
$\overline{Ie}$

Figure 1: Graphical representation of $F_Q(200)$.

possible combinations of $\bar{i}_e$ and t_{r_e} are accounted for; thus in deriving $F_Q(q)$, the probability of any combination $\bar{i}_e$-t_{r_e}, yielding $Q \leqslant q$, is explicitly accounted for.

3.3 SUMMARY OF THE CONCEPTUAL MODEL

Note, before studying its practical implications, that this model is very general.

- No assumptions are written relative to the probability density function $f_{\underline{\theta}}\ (\underline{\theta})$.
- No assumptions are imposed on the maximum number of random variables, $\underline{\theta}$, governing the rainfall-runoff process.
- No constraints are given to the selection of the rainfall-runoff model.

The conceptual model is also applicable to the derivation of frequency curves of other runoff statistics. For instance of the number and magnitude of overflows at any point in the drainage network, or of urban runoff quality indicators, provided a known and reliable transformation function, $Q(\underline{\theta})$ is available.

It is also interesting to discuss the design storm concept in locating this event in the $\bar{i}_e$-t_{r_e} plane. Normally its duration is equal to the time of concentration of the basin, $t*$, and its mean intensity is then determined from intensity-duration frequency (IDF) curves. This storm shown in figure 2 is a single point in the $\bar{i}_e$-t_{r_e} plane; its does not defines a contour $Q = q$ nor does it yield any information about the frequency of the runoff parameter.

The design storm approach is a design method which can be acceptable for design purposes; it cannot however rationalise any decisions or future interventions on the drainage system.

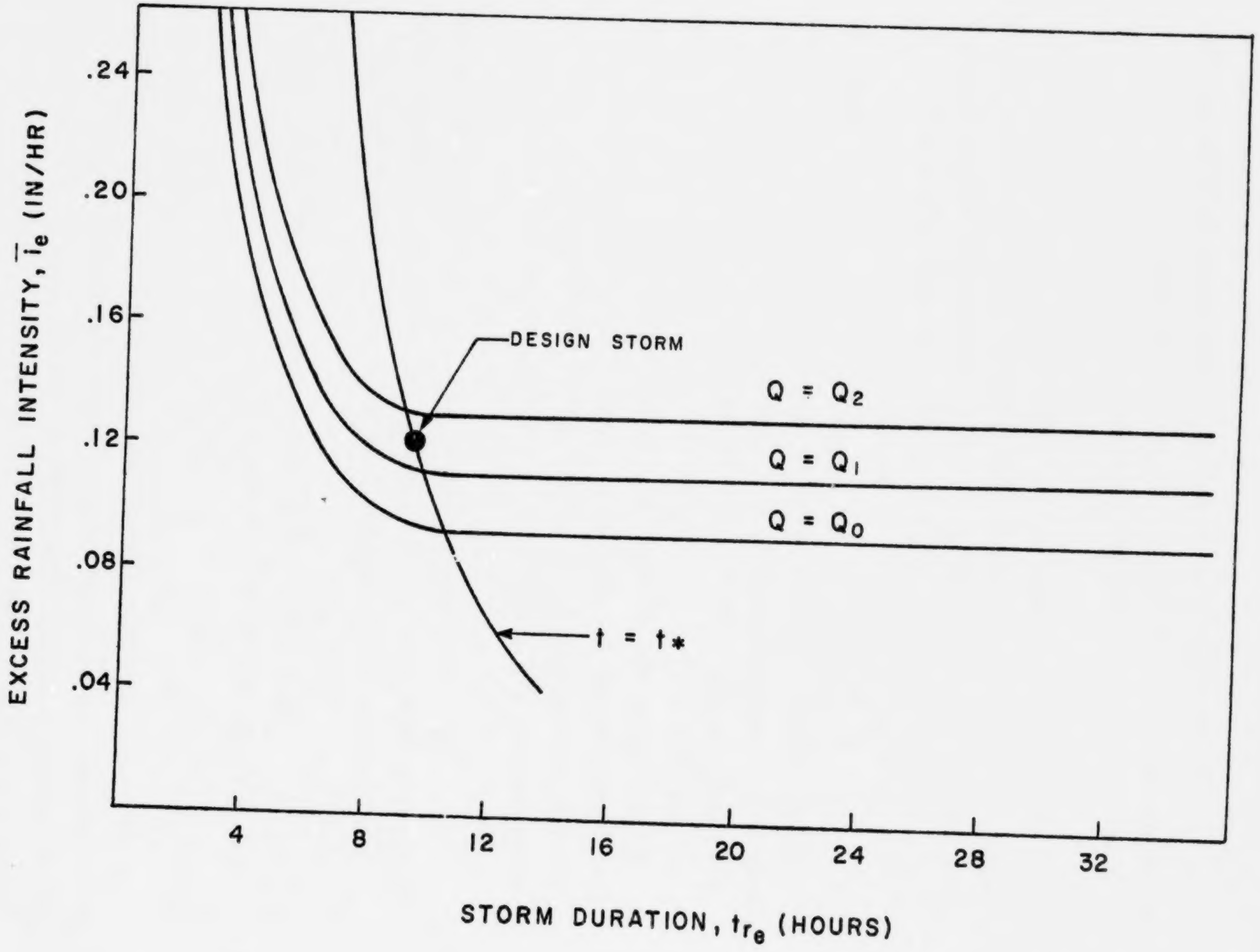

Figure 2: Location of the design storm in the t_e-t_{r_e} plane.

4.0 METHODS OF SOLUTION

Several numerical strategies can be designed to derive the frequency curves $T_E(Q)$ from random variables representative of the rainfall-runoff process. Leclerc and Schaake (1973) compared four possibilities including an analytical approach developed by Eagleson (1972) and a stochastic simulation method.

The stochastic simulation procedure is presented here but this does not imply that it is better than other approaches.

Figure 3 presents the solution procedure. Once selected the runoff statistic for which the frequency curve is sought, a transformation function, a rainfall-runoff model, is chosen.

A stochastic rainfall generator (the vector $\underline{\theta_2}$) is then selected. This choice is governed by several factors namely the availability of pertinent data, the presence of a regional meteorological station to be used as a pivot station, the size of the basin and the objectives of the study. For the revision of the drainage master-plan of Ville de Laval, rainfall statistics have been estimated for operationally independant rainfall events at the Dorval Airport meteorological station. In this the approach adopted by Eagleson was followed.

The rainfall generator then is used to simulate a trace of rainfall events; it contains a large number of events, most of them not of any interest for the frequency curve, since they yield values of q too frequent to be pertinent to the problem.

The purpose of the fifth step is the identification of those rainfall events whose runoff will be simulated. Design of the screening procedure of storm events is crucial and special attention must be given to it since any potentially interesting event not selected could never be included in the derived frequency curve. In its design consideration must be given to the nature of the runoff statistic and

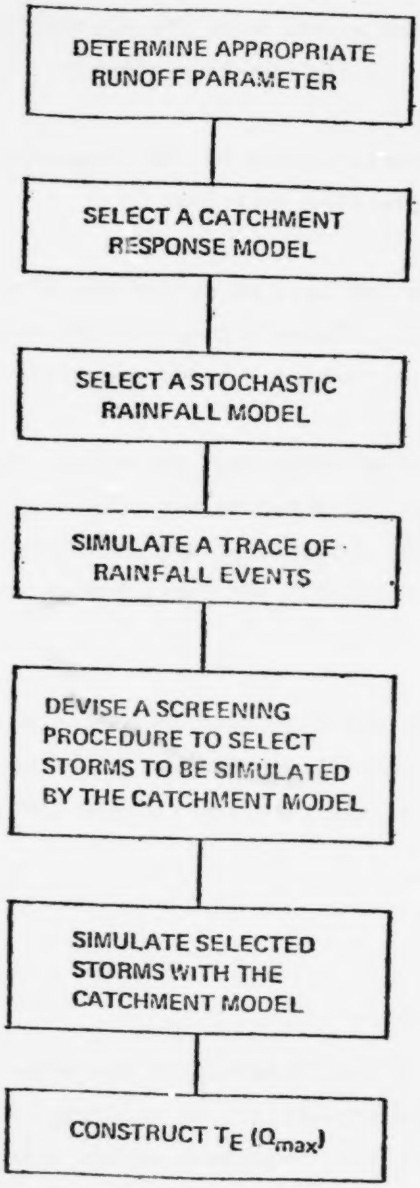
DETERMINE APPROPRIATE RUNOFF PARAMETER
SELECT A CATCHMENT RESPONSE MODEL
SELECT A STOCHASTIC RAINFALL MODEL
SIMULATE A TRACE OF RAINFALL EVENTS
DEVISE A SCREENING PROCEDURE TO SELECT STORMS TO BE SIMULATED BY THE CATCHMENT MODEL
SIMULATE SELECTED STORMS WITH THE CATCHMENT MODEL
CONSTRUCT $T_E(Q_{max})$

Figure 3: Simulation solution procedure for $T_E(Q_{max})$.

to the physiographic characteristics of the catchment. Upon complition of this step, a bank of rainfall events pertinent to the frequency curve sought is available. The next step consists in the simulation of these selected events with the rainfall-runoff model to determine the magnitude of the runoff statistic.

The final step, the construction of the frequency curve is done by assigning to each Q a plotting position.

Stochastic simulation was used to derive the flood frequency curve of natural catchments; figure 4 compares the analytical approach of Eagleson with the simulated flood frequency curve.

Stochastic simulation was also used to assess the relative efficiency of urban runoff control alternatives. Figure 5 compares peak runoff frequency curves for uncontroled and controled conditions. The control alternative is a small reservoir for which two outlet designs were studied.

Stochastic simulation was also used to estimate the size of a retention basin, storing runoff when discharges exceed a given threshold, QTH. Figure 6 relates the overflow storage and threshold discharge at constant T_E.

5.0 CONCLUSION

Frequency curves of a runoff statistic describe globally its range of variation and the probability of occurence of a given magnitude taken by the statistic. Frequency curves also allow relative assessment of the efficiency of control alternatives.

Additional effort is needed a) to develop numerically efficient methods of solution, b) to assess the component of the probability density function $f_{\underline{\theta}}(\underline{\theta})$ for urban drainage studies, as a function of the objective of each study, the size of the study area, etc., and

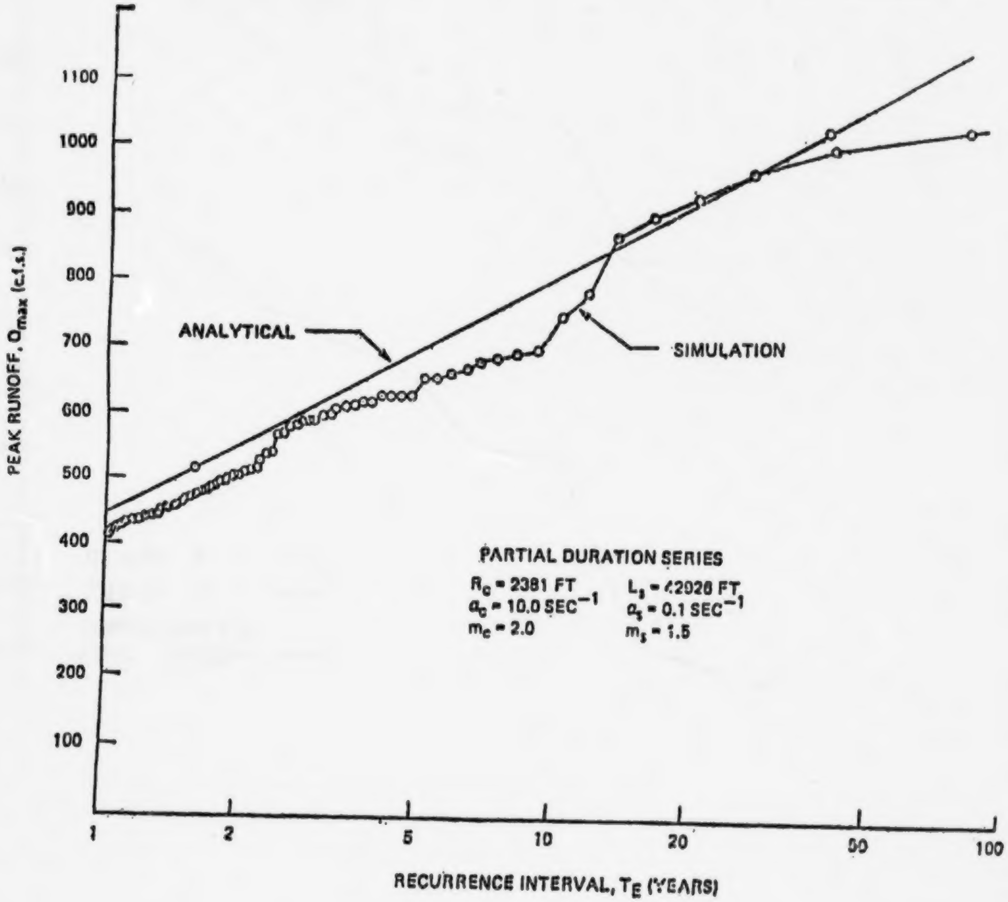

Figure 4: Analytical and simulated runoff frequency curves for the east branch of the Eightmile-River near North Lyme, Connecticut.

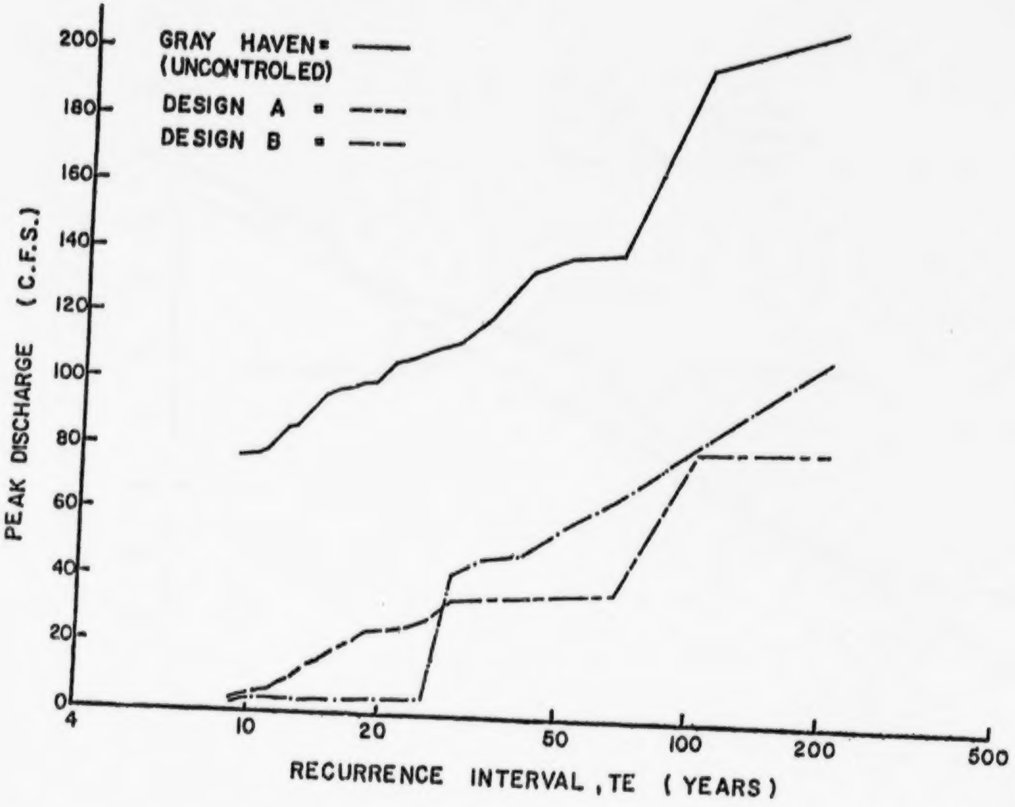

Figure 5: Peak runoff frequency curves for reservoir outlet designs A and B.

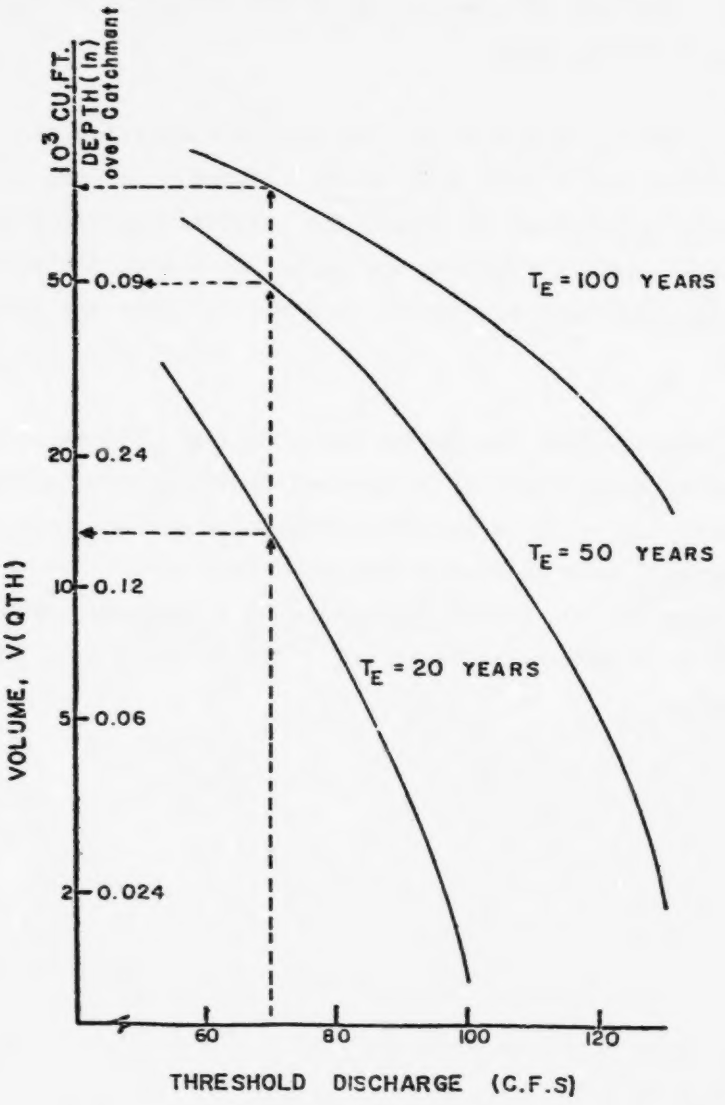

Figure 6: Overflow storage and maximum allowable outflow rate at constant T_E.

to c) establish $Q(\theta)$ when Q is a water quality variable of urban runoff or of the receiving stream.

The design storm plots as a single point in the $\bar{i}_e - t_{r_e}$ plane and does not yield any information about the frequency of occurence of the statistics of interest.

Stochastic simulation and the method suggesting the use of selected historical storms are closely related. In both cases a screening device is required to select the rainfall events of interest. Eventhough historical storms may appear more credible than stochastic simulation each approach has its own merits; they are indeed complementary.

It appears that the design storm concept will eventually be replaced by an ensemble of historical design storms; assessment of any runoff control alternatives or drainage designs will then be more complete than an evaluation based on a single storm event without calling for the formal derivation of a frequency curve. Such an approach will soon be proposed to Ville de Laval as a future design procedure.

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SUMMARY

SEMINAR ON THE DESIGN STORM CONCEPT

ECOLE POLYTECHNIQUE, UNIVERSITY OF MONTREAL

MAY 23, 1979[a]

PURPOSE OF SEMINAR

1. To sensitize civil engineers to the positive and negative features of the design storm concept so as to discourage its blind use.
2. To provide a forum for exchange of information and ideas among researchers in the United States, Canada, and elsewhere.

PANEL AND ATTENDEES

The nine person panel represented a variety of experience, interests, and employment situations. Panel members were:

M.B. McPherson, ASCE Urban Water Resources Research Program

Jiri Marsalek, Environment - Canada

Harry Torno, US EPA

Ben Urbonos, Urban Drainage and Flood Control District, Denver

Gilles Patry, Ecole Polytechnique, University of Montreal

Jacques Delluer, Purdue University, Indiana

Harry Wenzel, University of Illinois

Michael Terstriep, Illinois State Water Survey

Stuart G. Walesh, Donohue & Associates, Inc., Wisconsin

The seminar was chaired by M. McPherson. Approximately 40 people, in addition to the panel, participated in the seminar. Included in this group were foreign guests, students and staff of Ecole Polytechnique, and local engineers.

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[a] Presented at the May 24-25, 1979 SWMM Users Group Meeting in Montreal by Stuart G. Walesh, Head - Water Resources Section, Donohue & Associates, Inc., Wisconsin.

PROCEDURE

The seminar was initiated by providing each panel member with an opportunity to make a statement of ten minutes duration or less. Panel members used this as an opportunity to express concerns, present ideas, and comment briefly on some of their research and development efforts. The remainder of the seminar, which lasted for appproximately four hours, consisted primarily of response, questions and contributions from the audience. The seminar was characterized by an open exchange of ideas and information between the participants.

STRUCTURE OF THIS SUMMARY

Rather than attempt a sequential account of the ideas and information exchanged during the course of the seminar, this summary is structured so as to collect those ideas and information into related categories. More specifically, the following categories are described in detail below:

1. Definition of Design Storm.
2. Perceived or Demonstrated Short-Comings of the Design Storm Concept.
3. Perceived or Demonstrated Positive Aspects of the Design Storm Concept.
4. Alternatives to the Traditional Use of Design Storms.
5. Research Underway or Proposed.
6. Miscellaneous Considerations.

DEFINITION OF DESIGN STORM

The overall idea of a design storm is to provide a means of estimating a discharge or volume of specified recurrence interval for planning or design purposes. That is, a recurrence interval is assigned to the design storm, a rainfall-runoff procedure is used to convert the rainfall to runoff, and the recurrence interval of the design storm is transferred to the resulting runoff discharge or volume.

The general concensus of seminar participants was that there are two types of design storm.

Design storms are typically constructed so as to generate peak discharges. Therefore, design storms usually do not include the long duration-high volume rainfall events likely to be important in determining required sizes of detention/retention facilities. Design storms are even less likely to be suitable for determining non-point source pollutant loads to and concentrations in streams.

6. Areal variability of rainfall may be an imporant factor in determining discharge and volume at the catchment outlet. This variability is typically ignored in applying design storms.

7. Finally, engineers and other professionals involved in planning and design may expose themselves to criticism and even liability through use of an "unproven technique."

PERCEIVED OR DEMONSTRATED POSITIVE ASPECTS OF THE DESIGN STORM CONCEPT

The design storm concept enjoys widespread use. This is apparently in response to the following positive aspects of the concept.

1. Use of design storms requires minimal resources in terms of time and money.

2. Design storms appear to give conservative results, that is, high discharges and volumes.

3. Application of the design storm approach is generally accepted in practice and one can argue for maintenance of consistency of methodology in a given jurisdiction or geographic area.

ALTERNATIVES TO THE TRADITIONAL USE OF DESIGN STORMS

Seminar participants identified the following available or potential alternatives to use of design storms:

1. Continuous Simulation - Although this approach completely eliminates the need to use the design storm approach, it may prove to be costly primarily because of long computer runs. In addition, a continuous model may not be available to match the physical situation under study. Furthermore, precipitation data may not be available in the study area or may be of insufficient duration requiring excessive extrapolation of the resulting discharge-probability or volume-probability relationships.

2. Continuous simulation with cost-reducing techniques.

1. Direct use of intensity-duration-frequency (IDF) relationships to obtain a uniform intensity for a given duration and recurrence interval, such as is commonly done in applying the rational method of storm sewer design.

2. Development of a synthetic hyetograph (or storm profile) from IDF relationships, historic storms, or other means. Examples of design storms of this type are: the Chicago hyetograph and variations on it, the U.S. Soil Conservation Service Type II hyetograph, and the quartile hyetographs developed by the Illinois State Water Survey.

PERCEIVED OR DEMONSTRATED SHORT-COMINGS OF THE DESIGN STORM CONCEPT

1. A fundamental assumption associated with the design storm concept as it is usually applied is that the recurrence interval of the design storm may be transferred to the discharge or volume produced with the storm. This equivalency of recurrence interval has not been confirmed.

2. Casual observations of rainfall runoff data for watersheds reveal instances in which strikingly similar hyetographs produce markedly different hydrographs. Also, very similar hydrographs can be shown to have been generated by markedly different hyetographs. This suggests the important role of factors such as antecedent moisture conditions and areal distribution of rainfall over the catchment. Recent modeling studies (i.e., Marsalek-Canada, Wenzel and Voorhees-Illinois, and Urbonos-Denver) using historic storm series suggest that design storms may produce very large errors in calculated discharges.

3. Some of the methods used to construct design storms may be incorrect uses of IDF relationships. (i.e., possibility of incorrectly interpreting a Chicago-type hyetograph).

4. Design storms don't usually yield all the probability information desired for planning and design studies. For example, whereas a design storm might provide a flood flow of specified recurrence interval, it is not suitable for developing a flow duration curve. The need for additional probability-based hydrologic-hydraulic data is expected to become more pressing with increased work in the area of non-point source pollution.

5. If the design storm concept is generally less than adequate for determination of discharges of specified recurrence interval, then existing design storms are even less likely to be suitable for determination of volumes of specified recurrence interval.

3. Use of a series of "major" historic storms as input to an event model followed by discharge-probability or volume-probability analyses of the model output.

4. Improved, rationally developed and statistically meaningful design storms. One possibility is use of modeling with historic storm sequences to determine by "trial and error" combinations of hyetograph shape or peakedness (peak of hyetograph relative to duration), antecedent moisture condition, and duration that may be expected to yield accurate discharges or volumes of specified recurrence interval for a given geographic area and type of catchment. Another approach is based on the premise that a hyetograph is defined by the following five random, but not necessarily independent, variables: duration, mean intensity, temporal distribution, areal distribution, and antecedent moisture conditions. It may be possible to use probability theory to develop probability distributions for each of the five variables, to determine their interdependence, and to ultimately combine the distributions into design hyetographs.

5. Use of one or a few readily remembered historic storms for a study area without explicit assignment of a probability or recurrence interval. This may be an effective way to relate the carrying capacity of a plan or design to the public.

RESEARCH UNDERWAY OR PROPOSED

Given the variety of available alternatives to design storm concept, some of the seminar discussion was devoted to research that is underway or proposed. Research efforts identified during the seminar are as follows:

1. Use of a series of historic storms with an event model followed by subsequent discharge-probability or volume-probability analyses of the output. (i.e., Wenzel and Voorhees-Illinois, Marsalek-Canada, Urbonos-Denver and Walesh, Lau and Liebman-Wisconsin).

2. Reducing the cost of continuous simulation. (i.e., Geiger-Germany, Huber-Florida, Alley-USGS, and Walesh and Snyder-Wisconsin).

3. Probablistic development of a design storm. (i.e., Hydroscience for US EPA and Laurenson-Australia).

4. Use of modeling to identify appropriate combinations of hyetographs shape or peakedness, antecedent moisture conditions, duration, and areal availability. (For example, Urbonos-Denver, Lowing-United Kingdom, Falk-Sweden, and Geiger-Germany).

5. Development of computer programs intended to efficiently and effectively screen long-term historic or precipitation data to select subsets of storms suitable for simulation, with event models of discharges, volumes, or water quality. (For example, Terstriep-Illinois State Water Survey, Wenzel and Voorhees-Illinois, and Lowing-United Kingdom).

MISCELLANEOUS

Some of the ideas and information offered during the seminar do not fit into the above categories but are important and are presented as follows.

1. There seemed to be a general consensus that there never will be one methodology to replace the design storm concept but rather the goal should be an improved set of methodologies. Assuming the availability of such a set, practitioners could match methodologies to the characteristics of each particular planning or design project. In attempting to match methodologies to projects, questions such as the following should be raised:

   How complicated is the system?

   How sensitive is the size and cost of the planned system or the designed facility to the methodology?

   What are the likely consequences of failure?

2. It was suggested that we should perhaps separate our science from our engineering. Although many engineers are also scientists, it is important to recognize that a scientifically unsatisfying method may yield acceptable results for engineering purposes.

3. Land use changes may be expected to have a significant impact on the hydrologic response of a watershed. Expected impacts of urbanization should be accounted for in our hydrology either by conservatisms in hydrologic analyses or by forecasts of future land use.

4. Two apparently important characteristics of historic design storm hyetographs, as revealed by more recent investigations, are peakedness and areal distribution of precipitation over the catchment. Hyetograph peakedness should be a consideration in developing the hyetograph and in selecting the time increment for the hydrologic analysis. In a similar manner, possible areal

distribution of rainfall should be considered, particularly as larger and larger basins are analyzed.

5. Regardless of how the design storm issue is resolved, it is often necessary to determine discharges or volumes for two or more recurrence intervals for a given study catchment. For example, the design of a "minor" underground drainage system for a residential development, which may be sized to accommodate flows of a five year recurrence interval, should be followed by the design of a "major" above ground system, sized so as to accommodate discharges having a recurrence of perhaps 100 years.

6. Another important issue is the selection of the recurrence interval used in planning and design of storm and flood water facilities. A related issue is consistency of recurrence interval. For example, if residents of flood prone areas along major streams are provided with protection up to the 100-year recurrence interval flood stage, shouldn't similar levels of protection be provided to residential areas away from major streams that are subject to localized storm water inundation?

7. Although antecedent moisture conditions generally appear to have a significant influence on runoff discharges and volumes, there is some indication that this may not be as important a factor in arid areas.

8. As we proceed with our studies of the design storm concept, we should be cognizant of the ideal way to test the design storm concept and to develop alternatives to it. This will require long-term rainfall-runoff data on relatively small homogeneous catchments. If such data were available, it would be possible to do a parallel-type analysis. On one hand, historic rainfall data could be used to determine or develop design storm hyetographs which could, in turn, be used to develop a discharge-probability or volume-probability relationship for the catchment. Paralleling this, the long-term runoff data could be statistically analyzed resulting in a second discharge-probability or volume-probability relationship. The two discharge-probability or volume-probability relationships could then be compared and used to assess the validity of the design storm concept and to make adjustments to it.

CONCLUSION

One way to summarize the present status of the unproven design storm concept and of replacments for it is to say, as one participant did that "urban hydrology is a paradise for research." Perhaps that is an overstatement of the problem and the situation is not quite that gloomy.

On the positive side, the seminar revealed that both researchers and practitioners in the United States, Canada, and elsewhere are very much aware of the shortcomings of the design storm concept and are concerned about the problem. Equally important, they are exploring alternatives to and refinements of the design storm concept. Given that kind of atmosphere, we can be optimistic that much will be accomplished in the next few years.

USE OF RUNOFF/EXTRAN FOR HYDRAULIC ANALYSIS OF URBAN DRAINAGE SYSTEMS

For presentation at

SWMM USERS GROUP MEETING
Montreal, Canada

May 1979

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INTRODUCTION

This paper outlines some of our experiences in using the RUNOFF block and the EXTRAN block of the SWMM program. Primarily we have employed these programs for detailed hydraulic analysis of urban drainage systems.

We have based our mathematical modelling capabilities on RUNOFF and EXTRAN for two reasons. First, their basic approach to simulating the runoff and hydraulic transport processes are sound. Secondly, they are widely available and are non-proprietary. The last point is important in our opinion as the user has unrestricted access to the programs calculation methods.

Some readers may question our choice of the EXTRAN block instead of the TRANSPORT block. Most of our work is in the City of Winnipeg, which is located on extremely flat terrain. In general the ground level is about 20-feet higher than the normal water level of the Red River. Our sewer systems extend from 2 to 5 miles from the river and that sewer systems usually operate under surcharged conditions is a significant factor.

These programs have been criticized for their complexity. However, the processes they attempt to simulation are highly complex, and logically such programs will be complex also.

These programs are not totally comprehensive. In general, our philosophy has been both to adapt the programs to model the problem adequately where required and to simplify the problem to fit the program wherever possible. The expression equivalent pipe is frequently used. Consequently we have had to develop at least a working knowledge of computer programming and the operating system of the computer facility we use, in addition to an understanding of modelling techniques.

The SWMM programs were developed as a planning aid, specifically for pollution abatement analysis. When these programs are used for detailed hydraulic analysis in the design process, the user often faces both technical and logistical problems. As this is our primary interest in SWMM, we have found ourselves in the position of debugging, making minor corrections, and often modifying many of the quantity routines.

Computer development has made mathematical modelling techniques available to most hydraulic engineers. However, a modelling program cannot be used as a "Black Box", a pitfall which has caught most EXTRAN users to some degree. By employing a computer program the engineer cannot shift his professional responsibility to the program's authors or sponsors. It is the engineer's responsibility to understand how the program computes its answers, and their significance.

TECHNICAL PROBLEMS

We have had relatively few technical problems with the RUNOFF block. However, the magnitude and shape of the RUNOFF hydrographs it develops are critical in the design of sewer systems. Unless calibration data is available, an understanding of the method the program uses to compute hydrographs is essential to achieve reasonable results.

Through a literature review and detailed examination of the program listing, we were able to develop the sketch shown in Figure 1 which illustrates the program's methodology.

In contrast we have experienced numerous technical difficulties with EXTRAN. This program is extremely sensitive to the model configuration, stable results are often difficult to achieve. Problems with surcharge flow calculations and the resulting "Artificial Storage" require proper management. Again, after considerable research a conceptual sketch of Artificial Storage was developed as shown in Figure 2.

We have attempted to use the following special features: in line-storage nodes, weirs, orifices and tidal gates. These routines are probably adequate for qualitative work where hydraulic accuracy is not critical. However, in each case modifications to the program have been required to obtain correct hydraulic results for our purposes.

Our solutions to these problems and additional features we have developed are outlined later in the context of projects to which they apply.

NOTE: SUBCATCHMENT TREATED AS THREE SHEETS.

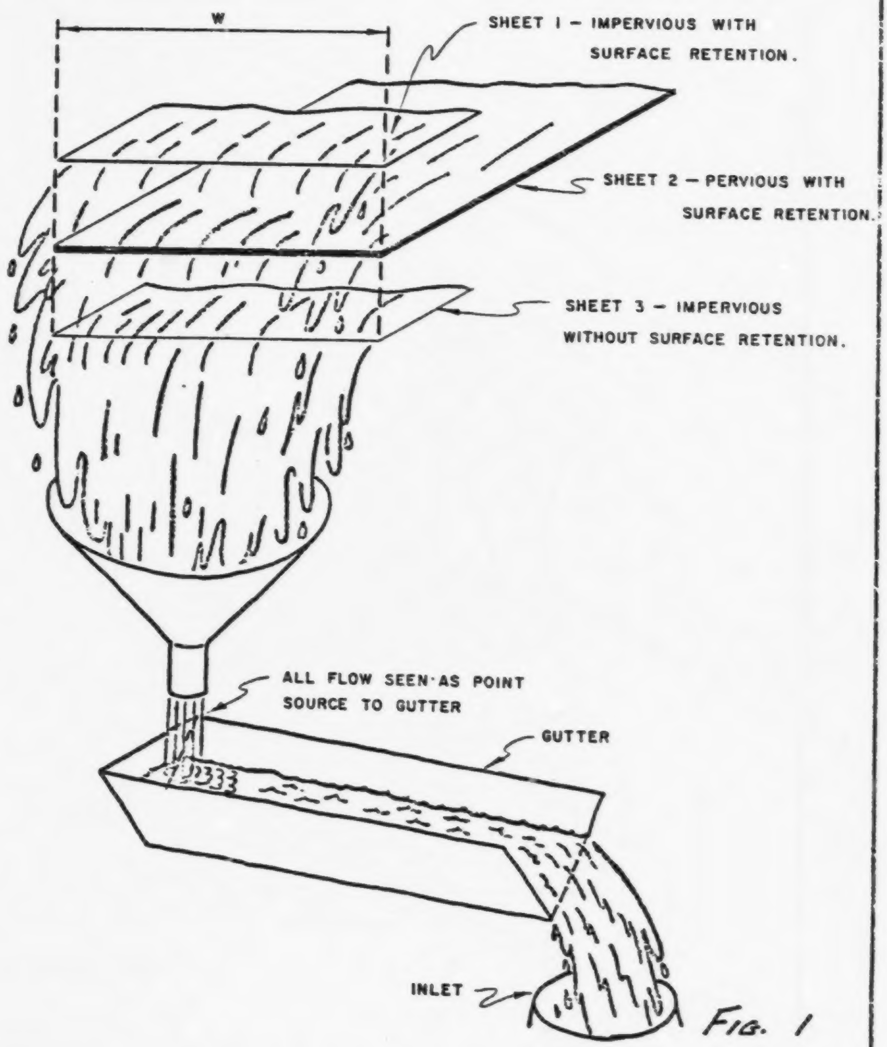
W
SHEET 1 – IMPERVIOUS WITH SURFACE RETENTION.
SHEET 2 – PERVIOUS WITH SURFACE RETENTION.
SHEET 3 – IMPERVIOUS WITHOUT SURFACE RETENTION.
ALL FLOW SEEN AS POINT SOURCE TO GUTTER
GUTTER
INLET

FIG. 1

TEMPLETON ENGINEERING COMPANY
PROFESSIONAL ENGINEERS
WINNIPEG MANITOBA

CONCEPTUALISED VIEW OF SWMM
RUNOFF BLOCK COMPUTATIONS

DATE: MAY / 79
SCALE: NONE
DWG. NO.

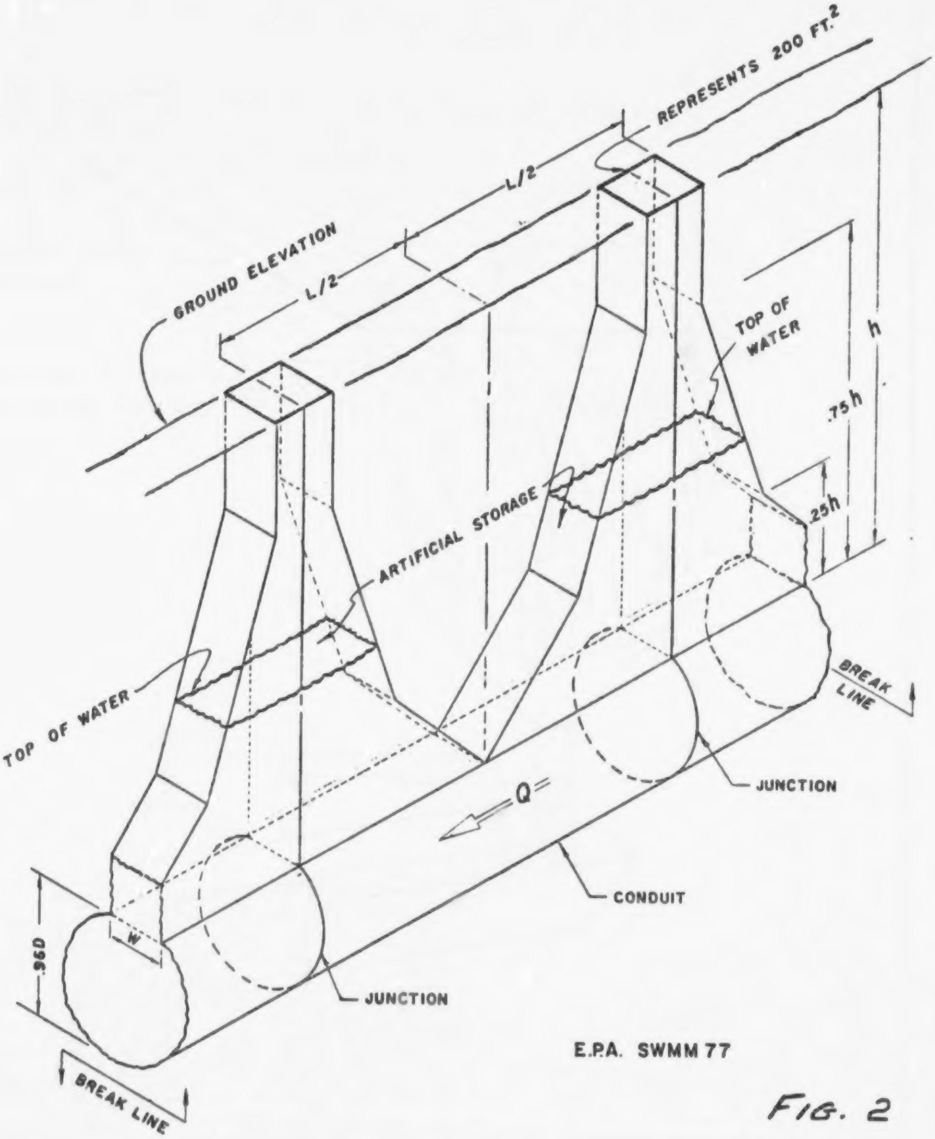
REPRESENTS 200 FT.²
L/2
L/2
GROUND ELEVATION
TOP OF WATER
h
.75h
.25h
ARTIFICIAL STORAGE
TOP OF WATER
BREAK LINE
JUNCTION
Q
CONDUIT
.96D
W
JUNCTION
BREAK LINE
E.P.A. SWMM 77

FIG. 2

TEMPLETON ENGINEERING COMPANY PROFESSIONAL ENGINEERS WINNIPEG MANITOBA	CONCEPTUALIZED VIEW OF SWMM EXTRAN BLOCK SURCHARGED FLOW COMPUTATIONS

DATE: FEBRUARY 8, 1978	SCALE: NONE	FIG. NO.

LOGISTICAL PROBLEMS

In our experience, the major logistical problems are:

1. The high cost of operating the programs, particularly EXTRAN. Our runs are generally in the \$40 to \$150 range.
2. The slow turn-around time associated with large programs, especially when using time-sharing facilities, a 24-hour turn-around is common for large runs.
3. The potential for errors to occur in the manipulation of larger volumes of input and output data.

Points 1 and 2 above are primarily caused by the size of these programs. For example, EXTRAN in SWMM 77 required in excess of 300K of computer core to operate.

For hydraulic analysis we use versions of RUNOFF and EXTRAN set up as individual programs, from which all quality routines have been removed. In addition, we have removed print-out associated routines and variable arrays which require core for storage, from EXTRAN. Our version writes the hydraulic results (flow and heads) for each cycle to a disk file, from which selected results are retrieved by a post-processor (a separate program--REPORT). As a result we operate RUNOFF at 145K and EXTRAN at 75K (100 pipes & 100 nodes). Turn-around times for EXTRAN average 20 minutes including a REPORT run. REPORT requires 101K to operate.

```
***************  COMPARATIVE TESTING RESULTS  *****************************
*
*                      EXTRAN - TECTRAN/REPORT
*                                                                         *
***************************************************************************
*          *            *              *               *                  *
*  NAME    * CM LWA+1   *  CPU TIME    *  SRUS USED    *                  ** SAVINGS *
*----------*------------*--------------*---------------*    JOBCOST   * FROM BASE*
*          *            *              *               *--------------*----------*
BASE RUN   *  226,667   *     22.693   *     45.311    *       $4.53  *    0.00% *
TEST 1     *  226,675   *      7.363   *     19.896    *       $1.99  *   56.08% *
TEST 2     *  125,435   *      7.800   *     17.481    *       $1.75  *   61.37% *
TEST 3     *   74,257   *      7.401   *     15.432    *       $1.54  *   66.23% *
***************************************************************************
*
* THE JOBCOST FOR A FULL REPORT RUN FOR THE ABOVE TESTS IS  $1.80.        *
*                                                                         *
***************************************************************************
```

NOTES:

(1) All versions handle 100 pipes & 100 nodes

(2) Base Run - Independent Program, no quality subroutines

(3) Test 1 - Printout Routines blocked

(4) Test 2 - Printout Subroutines & common block arrays removed (101 K)

(5) Test 3 - All remaining quality routines & arrays removed (51 K)

COMPARISON

	Base	Tectran/Report	Saving	Comments
Computation	$1.99	$1.54	23%	Fast Turnaround
Printing	$2.52	$1.80	29%	
	$4.53	$3.34	26%	All Computed Results Availabl

Fig 3

Figure 3 shows comparative core requirements and operating costs for various stripped down versions of EXTRAN.

The problems associated with the management of data, point 3 above, are two-fold. The output capabilities of EXTRAN are limited to 100 printed cycles, with Time Histories limited to 15 pipes and nodes. Therefore, the user has to select the output prior to making the computer run, which is a disadvantage since it is difficult to decide in advance which are the most important elements. Additional EXTRAN runs are often required to obtain complete results, especially when the model is unstable. Our post-processor, REPORT, has greatly improved this situation, through its ability of retrieving any computed data quickly from the disk file. Time histories can be obtained for any or all pipes and nodes and at intervals as close as every cycle, if desired. Examples of the new output are shown on Figures 4-6.

Input data manipulation problems were not so easily resolved as they involved an indefinable factor, the human element. Our approach has been to automate the creation of input files using interactive pre-processor programs for both RUNOFF and EXTRAN.

It is rarely practical to simulate large systems in a single run, a segmental approach is generally more efficient. For example, separate models for individual subtrunks and a model of the trunk alone. This results in

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```
1
                                                   P E A K   V A L U E S   R E P O R T
 ST. JOHN'S CALIBRATION  -  SUBTRUNK #08.
 GOVERNING TOP OF WATER OF ELEVATION 42.00 AT TRUNK REPLACED BY FREELY-                       FILE CREATION DATE/TIME  77/05/08.   11.55.25.


                                          PEAK NODAL INFLOW IN CFS. FOR   46 NODES.

   NODE  /  TIME     INFLOW   NODE  /  TIME     INFLOW   NODE  /  TIME     INFLOW   NODE  /  TIME     INFLOW   NODE  /  TIME     INFLOW
 ( 80110/02.00.10)    5.68  ( 80120/02.00.10)    1.20  ( 80121/02.00.10)    1.77  ( 80130/02.00.10)    4.86  ( 80140/03.00.00)    0.00
 ( 60000/03.00.00)    0.00  ( 60100/02.00.10)    1.43  ( 80000/03.00.00)    0.00  ( 80100/02.05.00)    4.58  ( 80101/02.00.10)    3.14
 ( 80200/02.00.10)    1.33  ( 80201/02.00.10)    4.10  ( 80202/03.00.00)    0.00  ( 80210/02.00.10)    4.02  ( 80220/02.05.00)    1.01
 ( 80230/02.00.10)    4.39  ( 80240/03.00.00)    0.00  ( 80300/02.05.00)    1.95  ( 80301/02.00.10)    3.01  ( 80302/03.00.00)    0.00
 ( 80310/02.00.10)    2.81  ( 80320/02.05.00)    1.87  ( 80330/02.00.10)    2.74  ( 80340/03.00.00)    0.00  ( 80400/02.05.00)    2.26
 ( 80401/02.00.10)    3.26  ( 80402/03.00.00)    0.00  ( 80403/02.00.10)    1.34  ( 80410/02.00.10)    3.16  ( 80420/02.05.00)    2.31
 ( 80421/02.05.00)    1.23  ( 80430/02.00.10)    3.13  ( 80440/03.00.00)    0.00  ( 80500/02.05.00)    2.26  ( 80501/02.05.00)     .72
 ( 80510/02.00.10)    3.20  ( 80520/02.05.00)     .95  ( 80521/02.05.00)     .79  ( 80530/02.00.10)    3.29  ( 80540/03.00.00)    0.00
 ( 80600/02.00.10)    3.34  ( 80700/03.00.00)    0.00  ( 90000/03.00.00)    0.00  (800000/03.00.00)    0.00  (600000/03.00.00)    0.00
 (900000/03.00.00)    0.00  (



                                  MAXIMUM NODAL WATER ELEVATION IN FEET FOR   46 NODES.

   NODE  /  TIME  WATER E.   NODE  /  TIME  WATER E.   NODE  /  TIME  WATER E.   NODE  /  TIME  WATER E.   NODE  /  TIME  WATER E.
 ( 60000/02.05.50)  37.66  ( 60100/01.59.20)  42.90  ( 80000/02.08.30)  41.89  ( 80100/02.08.20)  43.12  ( 80101/02.05.10)  46.83
 ( 80110/02.07.40)  46.33  ( 80120/02.07.40)  47.43  ( 80121/02.07.30)  48.51  ( 80130/02.04.20)  54.00  ( 80140/02.02.50)  54.06
 ( 80200/02.08.20)  45.13  ( 80201/02.06.40)  46.80  ( 80202/02.07.40)  46.81  ( 80210/02.07.20)  47.94  ( 80220/02.07.30)  48.91
 ( 80230/02.06.40)  51.47  ( 80240/02.06.40)  51.50  ( 80300/02.08.30)  47.36  ( 80301/02.08.40)  47.83  ( 80302/02.08.40)  47.80
 ( 80310/02.07.40)  48.68  ( 80320/02.07.40)  49.25  ( 80330/02.07.10)  49.68  ( 80340/02.08.10)  49.69  ( 80400/02.08.30)  48.43
 ( 80401/02.08.20)  49.74  ( 80402/02.08.40)  49.89  ( 80403/02.08.40)  49.97  ( 80410/02.03.40)  49.39  ( 80420/02.08.40)  49.76
 ( 80421/02.08.50)  50.07  ( 80430/02.03.40)  49.99  ( 80440/02.08.20)  50.07  ( 80500/02.08.30)  50.04  ( 80501/02.08.30)  50.08
 ( 80510/02.08.40)  51.22  ( 80520/02.03.50)  51.72  ( 80521/02.08.50)  51.76  ( 80530/02.08.50)  52.24  ( 80540/02.08.40)  52.26
 ( 80600/02.03.00)  50.70  ( 80700/02.08.00)  50.72  ( 90000/02.08.40)  41.96  (800000/02.08.40)  38.01  (600000/02.05.50)  35.13
 (900000/02.08.40)  38.11  (



                                          PEAK CONDUIT FLOWS IN CFS FOR   51 CONDUITS.

  CONDUIT/  TIME     FLOW   CONDUIT/  TIME     FLOW   CONDUIT/  TIME     FLOW   CONDUIT/  TIME     FLOW   CONDUIT/  TIME     FLOW
 ( 80100/02.08.00)  40.73  ( 80101/02.00.50)   2.57  ( 80110/02.04.00)   6.55  ( 80120/02.07.30)   3.10  ( 80121/02.07.30)  -2.64
 ( 80122/02.03.40)   2.01  ( 80130/02.06.50)   5.09  ( 80140/02.00.30)  -1.60  ( 80200/02.09.00)  29.84  ( 80201/02.01.00)   3.70
 ( 80202/02.07.00)   -.22  ( 80210/02.01.50)   6.33  ( 80220/02.00.00)   4.35  ( 80221/02.10.20)  -1.54  ( 80230/02.00.50)   3.54
 ( 80240/02.02.00)   -.66  ( 80300/02.09.10)  20.81  ( 80301/02.00.10)   2.18  ( 80302/02.05.50)   -.81  ( 80310/02.00.50)   5.13
 ( 80320/01.59.10)   4.65  ( 80321/02.10.00)   1.68  ( 80330/02.00.20)   2.45  ( 80340/02.07.30)   -.28  ( 80400/02.09.00)  14.33
 ( 80401/02.00.10)   3.42  ( 80402/02.01.00)   1.19  ( 80403/02.02.20)   1.44  ( 80410/01.59.30)   4.61  ( 80420/02.00.10)   3.67
 ( 80421/02.01.50)   1.28  ( 80430/02.01.00)   3.11  ( 80440/02.06.50)   -.96  ( 80500/02.09.00)   7.23  ( 80501/01.57.50)    .94
 ( 80510/02.00.40)   3.96  ( 80520/01.59.30)   3.13  ( 80521/02.01.00)    .95  ( 80530/02.00.00)   2.74  ( 80540/02.04.20)  -1.03
 ( 80600/02.06.30)   1.91  ( 80700/02.02.20)  -1.33  ( 90100/02.07.10)   4.79  ( 60100/02.01.20)   2.04  ( 60110/02.01.00)    .42
 (800000/02.08.40)  40.66  (600000/02.05.50)   1.43  (900000/02.08.40)   4.50  (      49/02.05.50)   1.43  (      50/02.08.40)  40.66
 (      51/02.08.40)   4.50  (
--EOR--
--EOF--
1
```

FIG. 4

```
                                  RUN DESCRIPTION

          ST. JOHN'S CALIBRATION  -  SUBTRUNK 808.
          GOVERNING TOP OF WATER OF ELEVATION 42.00 AT TRUNK REPLACED BY FREELY

          DISCHARGING PIPES DESIGNED TO CAUSE BACKUP TO ELEVATION 42.00 AT THE TRUNK.
          S.M.                                                               6-5-79.


                    INTEGRATION CYCLES      720

                    TOTAL NUMBER OF CONDUITS IS     51

                    TOTAL NUMBER OF NODES IS       46
                    LENGTH OF INTEGRATION STEP IS   10. SECONDS.

                    TIME ZERO IS 01.00.00.    TIME ENDED IS 03.00.00.


1                   T I M E  H I S T O R Y  O F  F L O W  A N D  V E L O C I T Y                    PAGE:   1.
                                   Q(CFS), VEL(FPS), HLOS(FT)
                                                                         FILE CREATION DATE/TIME  79/05/08.   11.55.25.
 ST. JOHN'S CALIBRATION  -  SUBTRUNK 808.
 GOVERNING TOP OF WATER OF ELEVATION 42.00 AT TRUNK REPLACED BY FREELY-

  TIME         CONDUIT  80100           CONDUIT  80101           CONDUIT  80110           CONDUIT  80120           CONDUIT  80121
            FLOW   VEL  ENT.  EXT.   FLOW   VEL  ENT.  EXT.   FLOW   VEL  ENT.  EXT.   FLOW   VEL  ENT.  EXT.   FLOW   VEL  ENT.  EXT.

01.05.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00
01.10.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00    0.00   0.0 0.00  0.00
01.15.00     .00    .2 0.00  0.00     .00    .5 0.00  0.00     .00    .4 0.00  0.00     .00    .2 0.00  0.00     .00    .1 0.00  0.00
01.20.00     .00    .3 0.00  0.00     .00   1.0 0.00  0.00     .01   1.1 0.00  0.00     .00    .4 0.00  0.00     .00    .3 0.00  0.00
01.25.00     .01    .5 0.00  0.00     .01   1.6 0.00  0.00     .05   1.8 0.00  0.00     .01    .6 0.00  0.00     .01    .4 0.00  0.00
01.30.00     .07    .9 0.00  0.00     .02   1.7 0.00  0.00     .07   2.1 0.00  0.00     .02    .8 0.00  0.00     .01    .5 0.00  0.00
01.35.00     .31   1.5 0.00  0.00     .02   1.9 0.00  0.00     .09   2.3 0.00  0.00     .03    .9 0.00  0.00     .02    .6 0.00  0.00
01.40.00     .72   1.9 0.00  0.00     .03   2.0 0.00  0.00     .12   2.5 0.00  0.00     .03    .9 0.00  0.00     .02    .6 0.00  0.00
01.45.00     .98   2.0 0.00  0.00     .04   2.2 0.00  0.00     .14   2.6 0.00  0.00     .03    .9 0.00  0.00     .01    .5 0.00  0.00
01.50.00    2.12   2.7 0.00  0.00     .19   3.5 0.00  0.00     .58   4.0 0.00  0.00     .10   1.2 0.00  0.00     .04    .7 0.00  0.00
01.55.00    8.89   4.1 0.00  0.00     .97   4.1 0.00  0.00    2.14   5.3 0.00  0.00     .34   1.7 0.00  0.00     .17   1.0 0.00  0.00
02.00.00   31.58   5.1 0.00  0.00    2.50   4.1 0.00  0.00    5.82   5.4 0.00  0.00    1.03   2.4 0.00  0.00     .38    .9 0.00  0.00
02.05.00   38.71   5.4 0.00  0.00    2.35   3.7 0.00  0.00    6.29   5.2 0.00  0.00    2.76   2.9 0.00  0.00   -1.97  -2.7 0.00  0.00
02.10.00   39.87   5.7 0.00  0.00    1.41   2.4 0.00  0.00    5.94   4.9 0.00  0.00    2.79   2.8 0.00  0.00   -2.19  -3.1 0.00  0.00
02.15.00   34.66   4.9 0.00  0.00    0.00   2.6 0.00  0.00    2.62   2.7 0.00  0.00     .55   2.6 0.00  0.00   -1.03  -1.9 0.00  0.00
02.20.00   30.72   4.4 0.00  0.00    1.73   2.4 0.00  0.00    1.81   1.6 0.00  0.00     .00    .9 0.00  0.00     .07    .4 0.00  0.00
02.25.00   27.18   4.1 0.00  0.00    0.00   2.6 0.00  0.00    1.53   0.0 0.00  0.00     .01    .5 0.00  0.00     .03    .5 0.00  0.00
02.30.00   21.37   4.0 0.00  0.00    0.00   3.0 0.00  0.00    1.41   0.0 0.00  0.00     .01    .6 0.00  0.00     .02    .5 0.00  0.00
02.35.00   14.82   3.7 0.00  0.00    1.18   0.0 0.00  0.00    0.00   0.0 0.00  0.00     .00   0.0 0.00  0.00     .02   0.0 0.00  0.00
02.40.00    9.56   3.4 0.00  0.00     .03   2.9 0.00  0.00     .79   2.8 0.00  0.00     .00    .8 0.00  0.00     .02    .6 0.00  0.00
02.45.00    7.32   3.3 0.00  0.00     .52   2.6 0.00  0.00     .47   2.7 0.00  0.00     .00   0.0 0.00  0.00     .01   0.0 0.00  0.00
02.50.00    6.16   3.2 0.00  0.00     .36   2.9 0.00  0.00     .35   2.4 0.00  0.00     .01   0.0 0.00  0.00     .01   0.0 0.00  0.00
02.55.00    5.44   3.1 0.00  0.00     .26   2.8 0.00  0.00     .34   2.5 0.00  0.00     .01    .7 0.00  0.00     .01    .5 0.00  0.00
03.00.00    4.92   3.0 0.00  0.00     .16   0.0 0.00  0.00     .32   2.7 0.00  0.00     .01    .7 0.00  0.00     .01    .5 0.00  0.00
--EOR--
--EOF--
1                   T I M E  H I S T O R Y  O F  F L O W  A N D  V E L O C I T Y
                                   Q(CFS), VEL(FPS), HLOS(FT)
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1                                       T I M E   H I S T O R Y   O F   H. G. L.
                                                  (VALUES IN FEET)                                                     PAGE:   2.

  ST. JOHN'S CALIBRATION  -  SUBTRUNK #08.                                                     FILE CREATION DATE/TIME  79/05/08.   11.55.25.
  GOVERNING TOP OF WATER OF ELEVATION 42.00 AT TRUNK REPLACED BY FREELY-

   TIME          GRND  52.80       GRND  52.00       GRND  52.00        GRND   0.00        GRND   0.00        GRND   0.00
              JUNCTION  60000   JUNCTION  80000   JUNCTION  90000    JUNCTION     0     JUNCTION     0     JUNCTION     0
  HH.MM.SS       ELEV   DEPTH      ELEV   DEPTH      ELEV   DEPTH       ELEV   DEPTH       ELEV   DEPTH       ELEV   DEPTH

  01.55.00      35.96    .39      37.87    1.00      38.09    .50
  01.56.00      36.00    .43      38.04    1.17      38.17    .58
  01.57.00      36.05    .48      38.26    1.39      38.28    .69
  01.58.00      36.12    .55      38.55    1.68      38.44    .85
  01.59.00      36.25    .68      38.93    2.06      38.65   1.06
  02.00.00      36.48    .91      39.44    2.57      38.84   1.25
  02.01.00      36.83   1.26      39.96    3.09      39.11   1.52
  02.02.00      37.16   1.59      40.28    3.41      39.34   1.75
  02.03.00      37.44   1.87      40.71    3.84      39.56   1.97
  02.04.00      37.58   2.01      41.07    4.20      39.91   2.32
  02.05.00      37.64   2.07      41.38    4.51      40.38   2.79
  02.06.00      37.66   2.09      41.63    4.76      40.91   3.32
  02.07.00      37.48   1.91      41.80    4.93      41.48   3.89
  02.08.00      37.27   1.70      41.88    5.01      41.89   4.30
  02.09.00      37.08   1.51      41.88    5.01      41.93   4.34
  02.10.00      36.86   1.29      41.81    4.94      41.64   4.05
  02.11.00      36.55    .98      41.68    4.81      41.15   3.56
  02.12.00      36.35    .78      41.53    4.66      40.64   3.05
  02.13.00      36.18    .61      41.33    4.46      40.20   2.61
  02.14.00      36.05    .48      41.13    4.26      39.84   2.25
  02.15.00      35.99    .42      40.96    4.09      39.57   1.98
  02.16.00      35.94    .37      40.80    3.93      39.35   1.76
  02.17.00      35.90    .33      40.66    3.79      39.17   1.58
  02.18.00      35.87    .30      40.53    3.66      38.97   1.38
  02.19.00      35.85    .28      40.41    3.54      38.65   1.06
  02.20.00      35.84    .27      40.29    3.42      38.50    .91
  02.21.00      35.83    .26      40.19    3.32      38.47    .88
  02.22.00      35.83    .26      40.10    3.23      38.41    .82
  02.23.00      35.82    .25      39.99    3.12      38.38    .79
  02.24.00      35.82    .25      39.89    3.02      38.35    .76
  02.25.00      35.81    .24      39.74    2.87      38.33    .74
  02.26.00      35.81    .24      39.59    2.72      38.30    .71
  02.27.00      35.80    .23      39.51    2.64      38.29    .70
  02.28.00      35.79    .22      39.40    2.53      38.26    .67
  02.29.00      35.79    .22      39.28    2.41      38.25    .66
  02.30.00      35.79    .22      39.18    2.31      38.24    .65
  02.31.00      35.78    .21      39.07    2.20      38.21    .62
  02.32.00      35.78    .21      38.97    2.10      38.21    .62
  02.33.00      35.77    .20      38.86    1.99      38.19    .60
  02.34.00      35.77    .20      38.77    1.90      38.18    .59
  02.35.00      35.76    .19      38.67    1.80      38.17    .58
  02.36.00      35.76    .19      38.57    1.70      38.16    .57
  02.37.00      35.76    .19      38.48    1.61      38.15    .56
  02.38.00      35.76    .19      38.39    1.52      38.14    .55
  02.39.00      35.75    .18      38.31    1.44      38.11    .52
  02.40.00      35.75    .18      38.24    1.37      38.12    .53
  02.41.00      35.75    .18      38.18    1.31      38.13    .54
  02.42.00      35.75    .18      38.13    1.26      38.11    .52
  02.43.00      35.74    .17      38.09    1.22      38.10    .51
  02.44.00      35.74    .17      38.05    1.18      38.10    .51
1                                       T I M E   H I S T O R Y   O F   H. G. L.
                                                  (VALUES IN FE)ET)
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multiple models and their associated input decks for a single system. Identifying and filing this data becomes a major problem. Our pre-processors are designed to retrieve input data from master files which contain all pertinent data, such as all pipes and nodes for the entire system. Only the master files are saved as any input deck can be recreated quickly and accurately without any re-coding at all. The EXTRAN pre-processor will also accept new or modified data, which can be used to create or update master files using ancillary routes.

A flow chart of our operating system is shown in Figure 7. It appears to be complex but it eliminates a great deal of the routine "bull work". However, it does demand a higher level of user competence than is required to fill out coding sheets by following the SWMM manual.

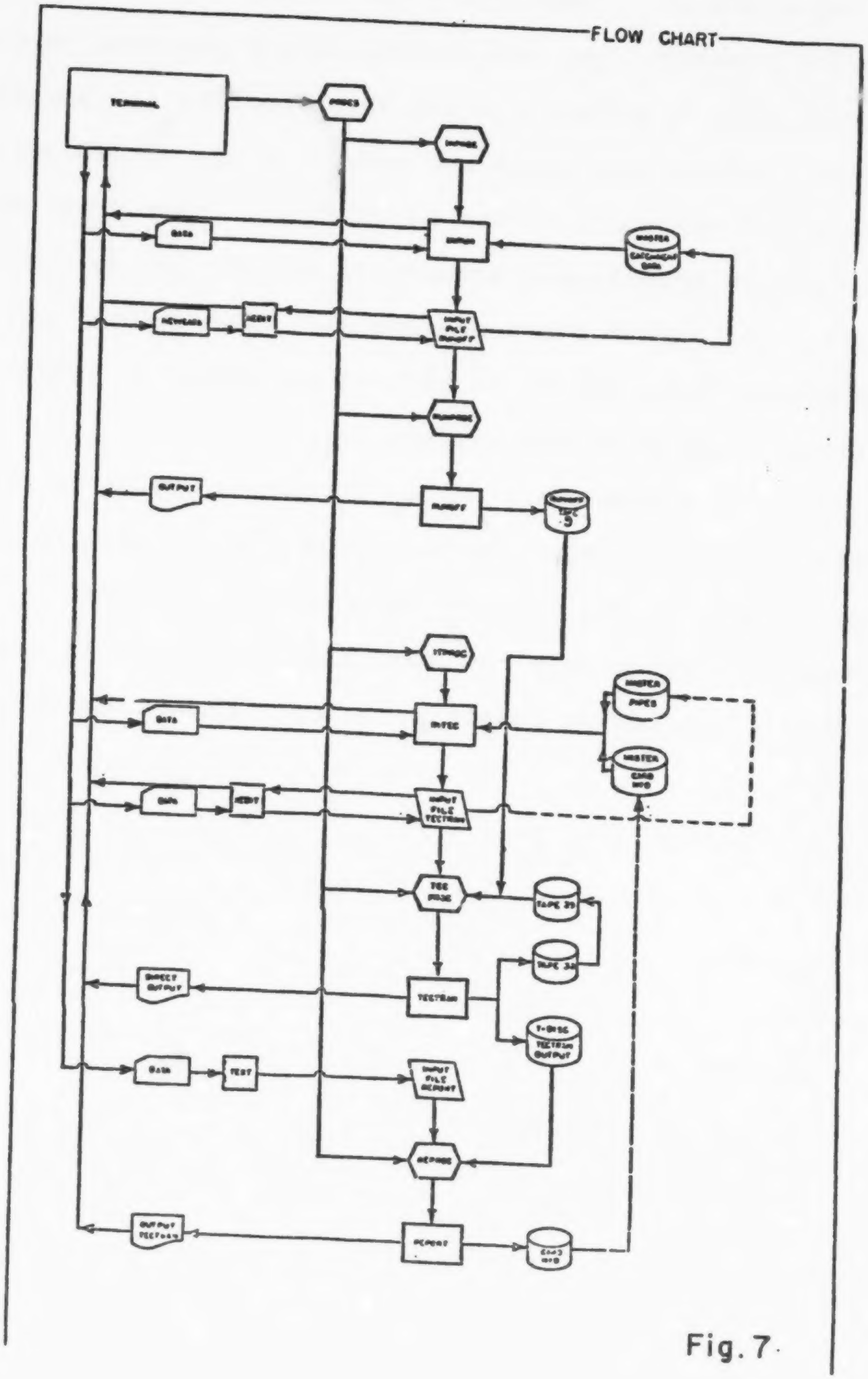

FLOW CHART
TERMINAL
DATA
INPUT FILE RUNOFF
OUTPUT
RUNOFF
TAPE 5
ITPROC
MASTER PIPES
MASTER COND INFO
DATA
INTEC
DATA
NEDIT
INPUT FILE TECTRAN
TEC PROC
TAPE 33
TAPE 34
DIRECT OUTPUT
TECTRAN
T-DISC TECTRAN OUTPUT
DATA
TEST
INPUT FILE REPORT
REPROC
OUTPUT TECTRAN
REPORT
COND INFO

Fig. 7

INLINE STORAGE NODES, ORIFICES/WEIRS AND LOOPED PIPES -- LEILA NORTH LAND DRAINAGE SYSTEM

In 1978 we carried out a preliminary engineering study, and the detailed design for the first phase, of a land-drainage system for a 700-acre residential development in the north west sector of the City of Winnipeg. The site is located a little over two miles from the Red River, the nearest discharge point, with a fully developed residential area inbetween.

A planning study, by Templeton Engineering Company, in 1974, established the possibility of using detention storage lakes which will discharge, off-peak, into an adjacent land drainage trunk sewer.

The system will be quite complex for a land drainage system. There will be three or more large storage lakes (design storages totalling 4.5 million ft^3), reversing pipe flows and an interconnected pipe system with two discharge points. It was this complexity that encouraged us to use EXTRAN for the hydraulic analysis of the proposed system. However, we were involved in a considerable research effort to develop the use of in-line storage nodes and their required weirs and orifices.

A schematic of the initial model of the system is shown in Figure 8). The model will become more complex as the system develops.

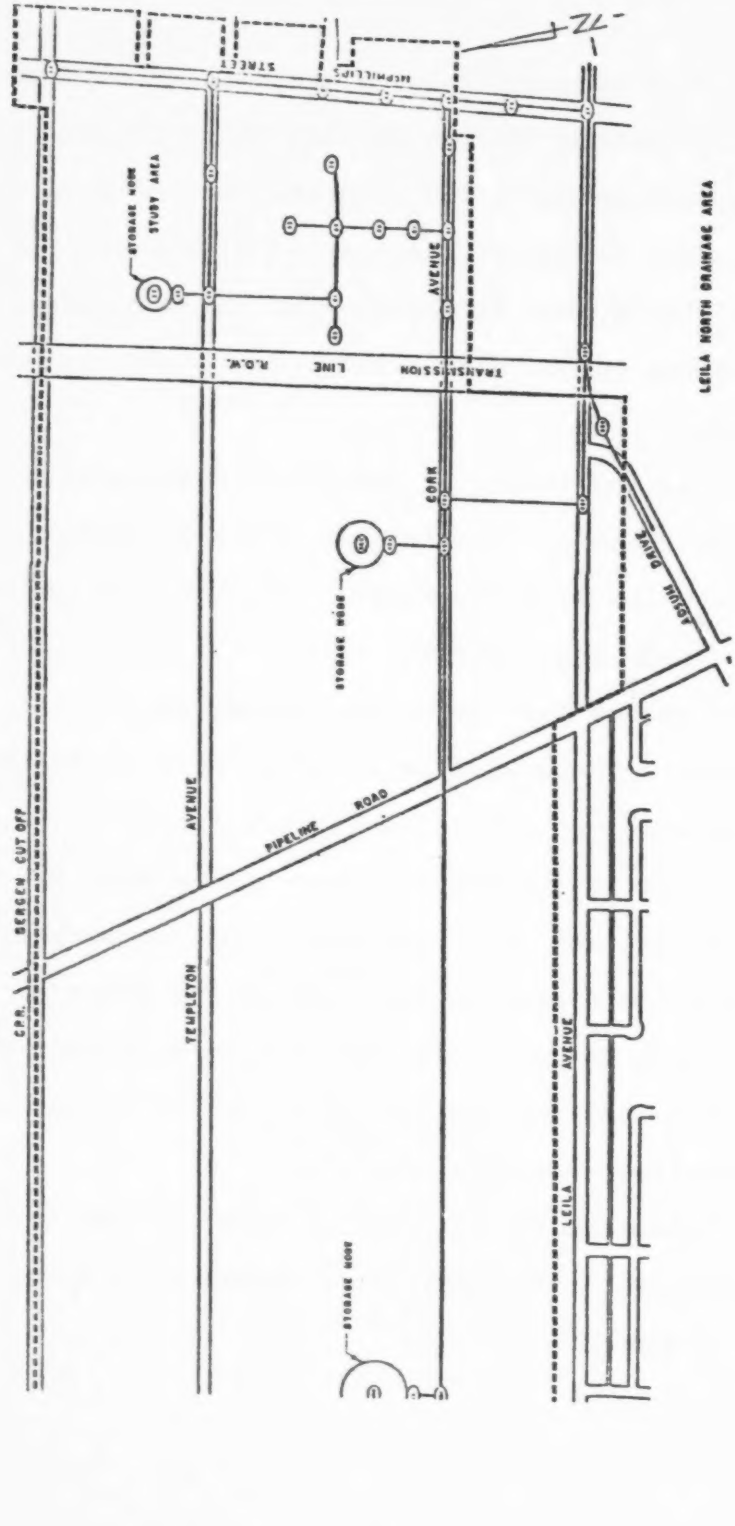

STORAGE NODE
STUDY AREA
McPHILLIPS STREET
CORK AVENUE
TRANSMISSION LINE R.O.W.
LEILA NORTH DRAINAGE AREA
STORAGE NODE
ADSUM DRIVE
PIPELINE ROAD
C.P.R. BERGEN CUT OFF
TEMPLETON AVENUE
LEILA AVENUE
STORAGE NODE

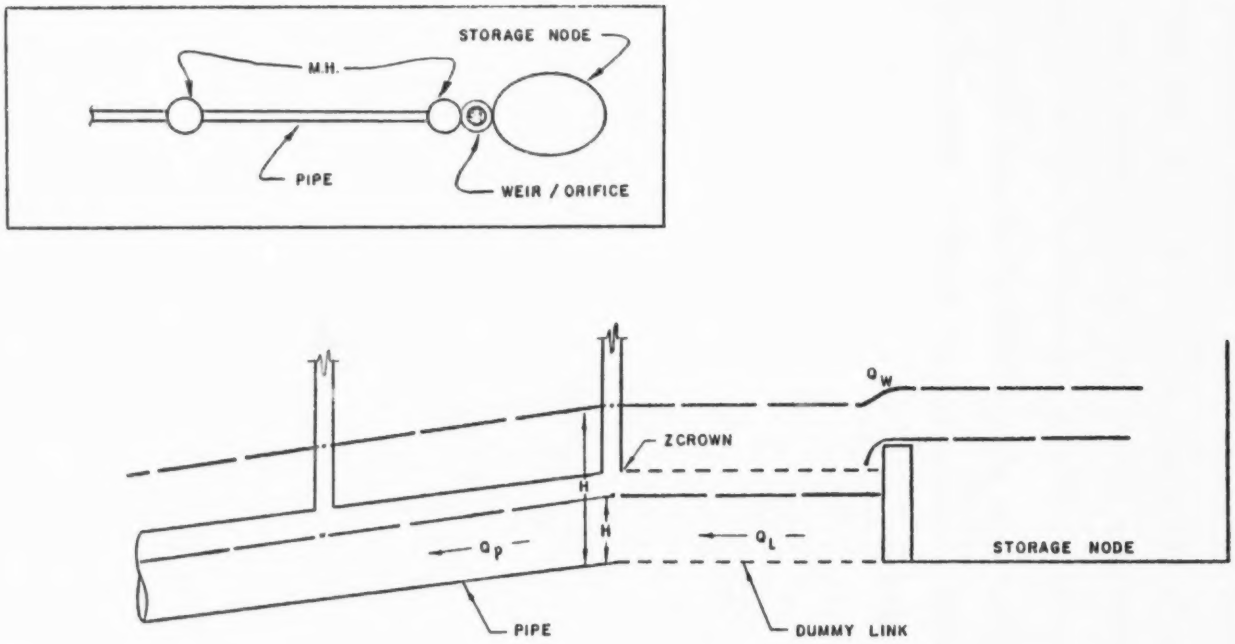
Fig. 8

The pipe system is sized to carry a 5-year return rainfall, while the storage facilities are designed to contain a 25-year-rainfall within a 4-foot rise above normal water level. Initial EXTRAN simulations were highly unstable. We found that the instabilities originated in the flow transfer devices, weirs or orifices, required to link the storage node to the pipe systems. Stable runs were achieved by using very large orifices with their inverts set at the normal water levels of the storage lakes. Under partial flow conditions EXTRAN computes orifice flows by a weir equation. Although this produced stable results they were not entirely correct. Under conditions where the lakes were discharging, it appeared that the mass discharge through the orifice (Q x t) did not match the rate of change of water level in the storage nodes as computed from the printed results. By getting print out cycle-by-cycle we found that what was actually happening was that discharges were pulsating.

Our solution was to develop our own transfer device, a Control Weir, which simulates the operation of the actual system that will control the discharge from the storage lakes. The lake will discharge over a weir, which maintains the normal water level of the lake. The downstream discharge pipe is installed such that its crown is at or close to the weir crest elevation. This allows relatively high discharges at low storage levels in the lakes, when flows are low in the downstream pipe system. At high lake levels the discharge is controlled by the discharge capacity of the outfall pipe or backwater effects of the downstream pipe system. The latter can reverse flow into the storage lake. Except for very low flows the discharge is controlled by the discharge pipe and the weir is in effect only a minor restriction.

Our Control Weir works as shown in Figure 9. The weir discharge is computed using the weir equation, but the flow transfer to the downstream node is accomplished using the simpler dummy link concept used by the existing orifice routine. However, if the storage node is discharging and the discharge pipe is surcharged, the flow in the dummy link (Q_L) is not allowed to exceed the flow in the discharge pipe (Q_p). If this occurs Q_L is reset equal to Q_p or a user specified percentage of Q_p. Since Q_L is reset before the elevation changes are computed at the nodes, mass balances within the model are maintained. This aids in ensuring that the simulation will not become unstable. Once we inserted this routine into the BOUND subroutine, our model simulations worked satisfactorily.

This is an example of what we call a "site specific modification". We know it worked for this particular problem, but it may not be generally applicable to all such problems. However, the basic approach used could be employed to resolve similar modelling problems.


STORAGE NODE
M.H.
PIPE
WEIR / ORIFICE
Q_W
ZCROWN
H
H
Q_P
Q_L
STORAGE NODE
PIPE
DUMMY LINK

$$\text{IF} \left[\begin{array}{l} Q_P \ +\text{VE} \\ Q_L > 0 \\ H > \text{ZCROWN} \\ Q_L > (f)\,Q_P \end{array} \right] \quad Q_L = (f)\,Q_P$$

CONTROL WEIR

Also of interest is the method used to insert this modification into the program, which reduced the amount of reprogramming required. However, this was at the expense of making the program a little more complicated to operate while using this routine. In brief, control weirs are coded as orifices in the input data deck; a single card entered in the EXECUTIVE block enables the program to differentiate between orifices and control weirs. This limited the modifications to the EXECUTIVE and BOUND subroutines and avoided changes to the INDAT 1 subroutine and the common block arrays.

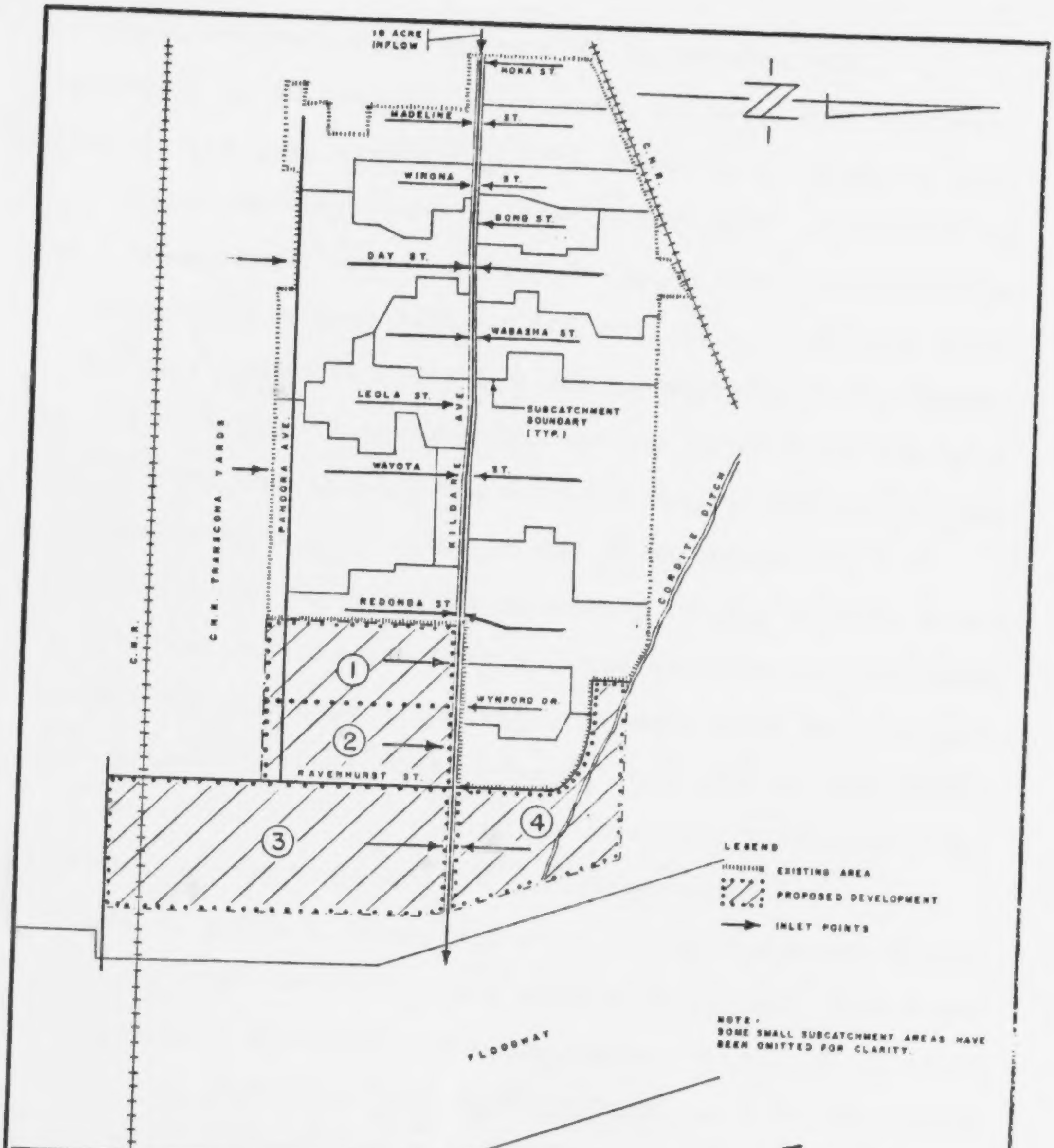
UNCALIBRATED SWMM AS A PLANNING AID -- KILDARE LAND DRAINAGE TRUNK SEWER ANALYSIS

The purpose of this study was to determine the effect on the level of protection provided by an existing land drainage trunk sewer if its drainage area was increased by 550 acres. The system, was at that time, servicing approximately 1450 acres of typical urban development. The trunk ranges in size from 54" to 114" over a 2 1/2 mile length and discharges to the Red River Floodway. It was proposed to connect the new development drainage at the very downstream end of the trunk as shown on Figure 10.

In addition to the directly connected system this trunk carries pumped discharge from a detention pond system which will ultimately service 2800 acres. The design and operation of this detention system is such that it did not affect the problem that we were attempting to resolve in this particular analysis.

The existing system was designed by the rational method using a rainfall intensity formula which is now obsolete. There was no data available with which to calibrate a simulation model and, also, time and financial restraints did not permit a detailed analysis.

We employed the SWMM program as a planning aid by using it to determine the hydraulic capacity of the trunk when servicing only the existing area and computing the effect on this capacity if the new area were also serviced.


18 ACRE INFLOW
HOKA ST.
MADELINE ST.
WINONA ST.
BOND ST.
DAY ST.
WABASHA ST.
LEOLA ST.
SUBCATCHMENT BOUNDARY (TYP.)
WAVOTA ST.
REDONDA ST.
WYNFORD DR.
RAVENHURST ST.
KILDARE AVE.
PANDORA AVE.
C.N.R. TRANSCONA YARDS
C.N.R.
C.N.R.
CORDITE DITCH
FLOODWAY
①
②
③
④
LEGEND
EXISTING AREA
PROPOSED DEVELOPMENT
INLET POINTS

NOTE:
SOME SMALL SUBCATCHMENT AREAS HAVE BEEN OMITTED FOR CLARITY.

FIG. 10

TEMPLETON ENGINEERING COMPANY
PROFESSIONAL ENGINEERS
WINNIPEG MANITOBA

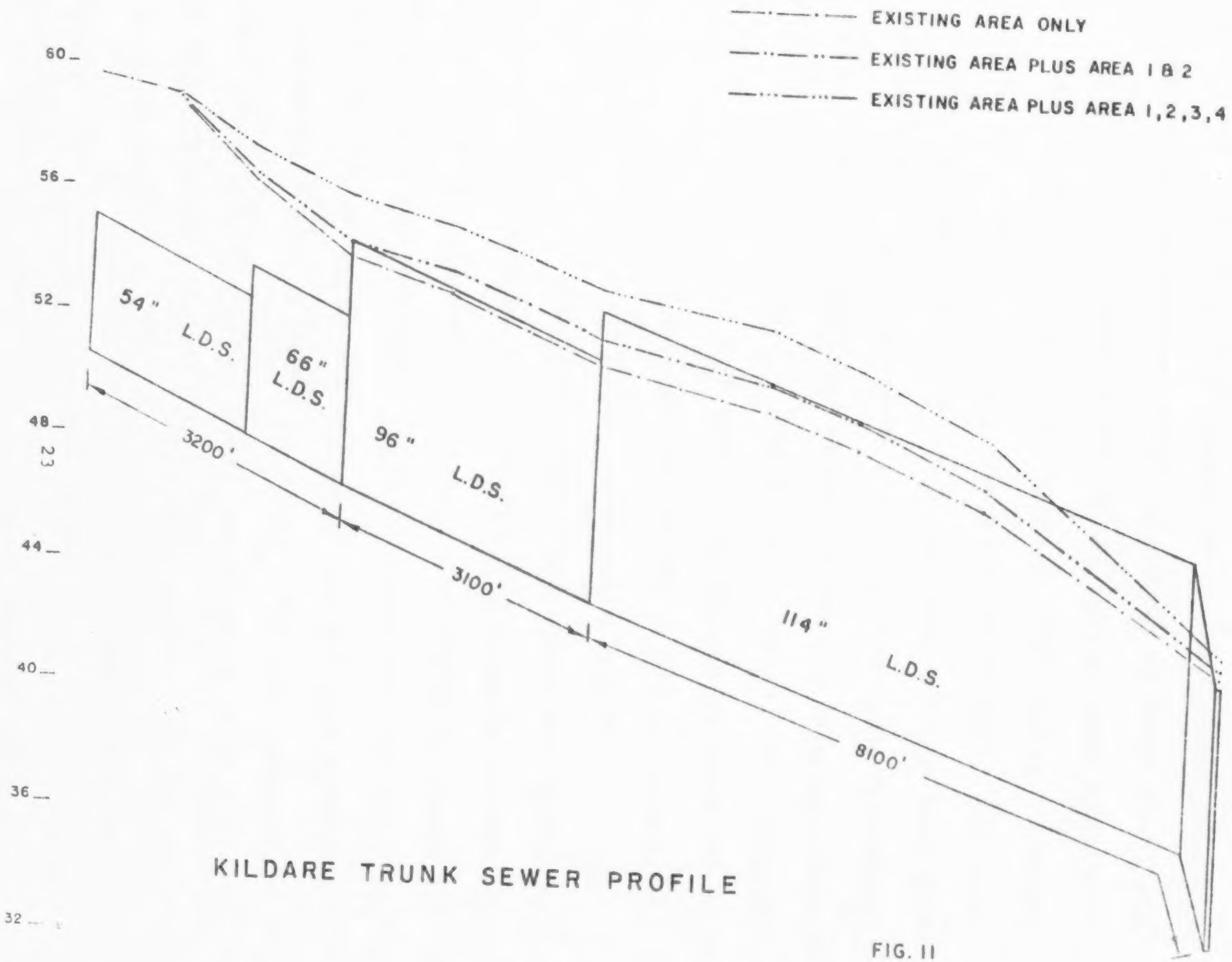
KILDARE LAND DRAINAGE SYSTEM
COARSE MODEL

DATE: MAY, 1978.
SCALE: 1 ≏ 2000

It was not possible, and fortunately not necessary, to determine the level of service, in terms of return period, represented by the hydraulic capacity.

We decretized the drainage area into coarse (lumped) subcatchments as shown on Figure 10. The RUNOFF block was used to generate inlet hydrographs of reasonable shape and peak values while the EXTRAN block was used to route these flows through the trunk sewers. Since only the trunk pipes were analyzed in EXTRAN, the model was balanced by checking to ensure that the subtrunks could actually deliver their computed peak flows to the trunk against the water levels computed for their inlet points. An iterative procedure was used in which runoff rates were increased until computed inflows could just be delivered to the trunk by the subtrunks.

It was found that, although inflows from the proposed new areas were added very close to the outfall, backwater effects did extend for a considerable distance upstream as shown in Figure 11. However, these backwater effects were dissipated before reaching the high end of the trunk and did not restrict the capacity of the high end subtrunks. For the length of trunk in which backwater effects were significant their effect on subtrunk capacity was minimal. These comparative results were checked by subsequently reducing and increasing the magnitudes of the inlet hydrographs through reasonable ranges and repeating the EXTRAN computations. In each case, backwater effects


EXISTING AREA ONLY
EXISTING AREA PLUS AREA 1 & 2
EXISTING AREA PLUS AREA 1,2,3,4
60
56
52
48
23
44
40
36
32
54" L.D.S.
66" L.D.S.
96" L.D.S.
114" L.D.S.
3200'
3100'
8100'

KILDARE TRUNK SEWER PROFILE

FIG. 11

did not reach the high end of the trunk and their influence on affected subtrunks was not significant. We, therefore, concluded that the trunk could safely service the new development without significantly reducing the level of service to the existing area.

This study was a good example of how SWMM can be efficiently and economically used in the planning/ decision-making process.

INITIALIZATION, MASS BALANCED STREET/BASEMENT FLOODING AND FLAP GATES -- THE ST. JOHN'S-POLSON COMBINED SEWER SYSTEM

This project involves the design of relief measures for two adjacent combined sewer systems, St. John's and Polson, and probably part of a third system, Jefferson, which borders them on their high ends. The total catchment area of the three systems is in the order of 1800-1900 acres. These areas suffer extreme basement flooding in response to the intense thunderstorms common during the summer months. The normal summer river level is elevation 734 (geodetic), with most basement elevations slightly over 750 (some as low as 747), the system has only a 16-foot operating head.

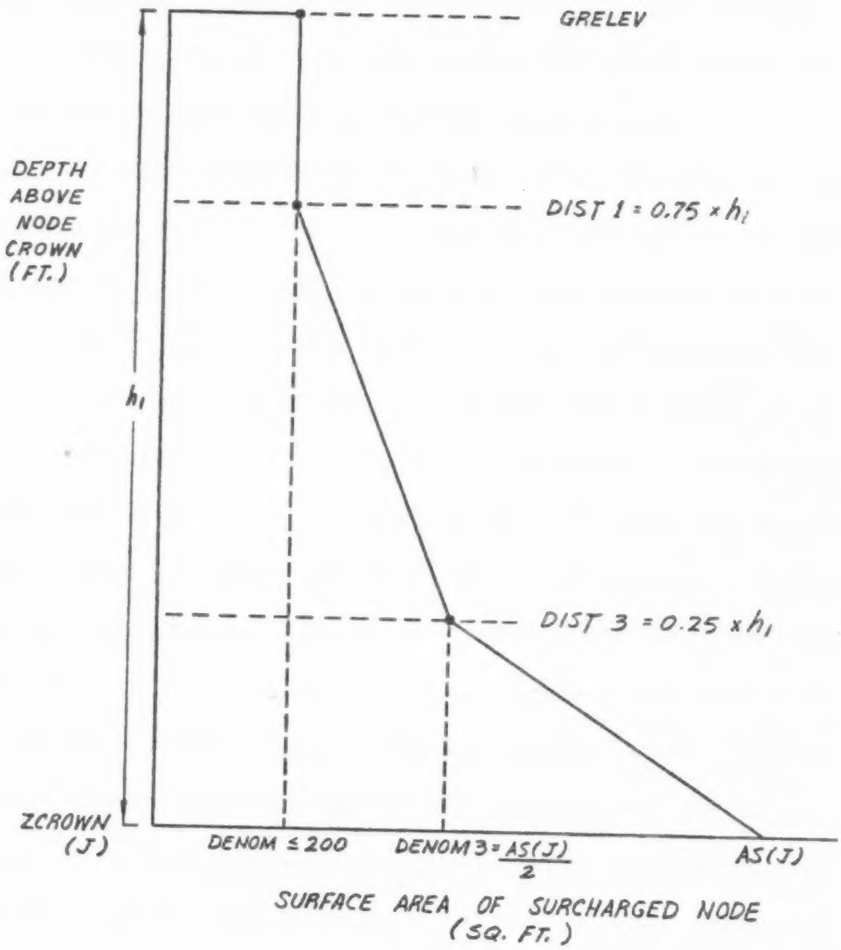
This is by far the biggest and most complicated project which we have tackled so far. A great deal of our development work and most of our attempts to improve the accuracy of the program have been pursued with projects like this in mind. Our pre-processor and post-processor programs were designed with the objective of making multiple simulations for just such a project practical.

A reasonably refined model of the St. John's system alone employs more than 350 pipes. Possible relief system integrating all three systems will involve 700-900 pipes to be analyzed. Because of the size and complexity of this sytem, it will be impractical to model the complete system in a single run. Basically, we are analyzing the individual subtrunks and the trunk separately using EXTRAN.

Initially the trunk systems are being analyzed using coarse subcatchment models. Subsequently, these will be refined using card hydrographs produced from detailed subcatchment models. These subcatchments are run with either an equivalent outfall pipe simulating the trunk response or a constant water level where appropriate. The labour and time required for this procedure is greatly reduced by automating the input data including the card hydrographs.

Two purely technical routines which we have added to our program will play an important role in this project. One is an initialization routine through which we will be able to tackle the problem of "Artificial Storage" affecting the computation of surcharge flows. Our basic approach here is to reduce the amount of artificial storage available in the model. Figure 12 illustrates the artificial storage curve in SWMM 77. This results in a more unstable model, which can only be stabilized by reducing the time step. However, reducing the time step, and in some cases this will be a drastic reduction, increases the costs of operating the model. These operating costs are almost directly proportional to the number of cycles in each simulation. By employing our initialization routine, we are able to change the time step (DELT) by stopping and restarting a simulation, and only using small time steps when necessary to achieve stable results.

This initialization is achieved by directing the program to write the values of certain internal variables to


GRELEV
DEPTH ABOVE NODE CROWN (FT.)
DIST 1 = 0.75 × h_1
h_1
DIST 3 = 0.25 × h_1
ZCROWN (J)
DENOM ≤ 200
DENOM 3 = $\frac{AS(J)}{2}$
AS(J)
SURFACE AREA OF SURCHARGED NODE (SQ. FT.)

REDUCTION OF AS(J) UNDER SURCHARGE
IN SWMM 77

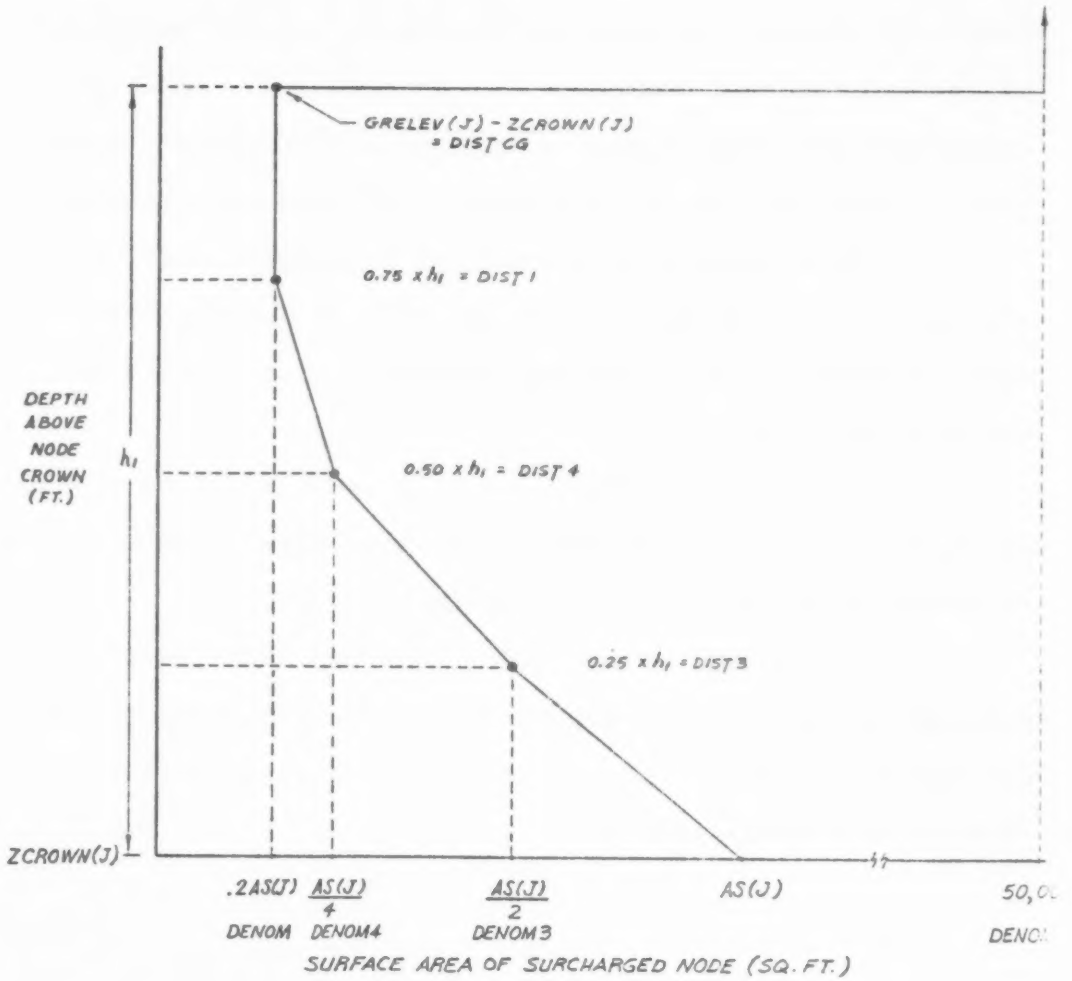
a disk file at a predetermined cycle. A subsequent run is then started at this cycle by reading in the disk file, which resets the variables for the initial cycle, prior to the execution loop in TRANQT. The program and input deck are loaded in the normal manner, only the number of cycles (NTCYC) and the time step (DELT) being changed. Apart from choosing the correct variables to initialize, the most difficult problem was to make subroutine INFLOW read the correct hydrograph values to match the new start time.

The other technical innovation is a routine developed so that we can simulate street or basement flooding. In conjunction with this routine, we also compute and print out the internal storage volumes employed in a particular simulation. We recognize three types of internal storage; real storage in the pipes, the artificial storage, which is part of the program methodology for surcharge conditions, and the real storage which is available either in the basements of combined sewer systems or on the streets of separate storm sewer systems. This facility makes it possible to calibrate a model to a runoff event which causes basement flooding in the system.

These flooding storage calculations were implemented by modifying the surcharge calculations of EXTRAN to prevent water loss from the system when the hydraulic gradeline reaches the ground or basement levels that are simulated.

This capability is achieved by adding another deflection point to the Artificial Storage reduction curve at the ground level, GRELEV, as shown in Figure 13. The surface area DENOM is made large (50,000 ft^2) above this point and the water depth Y is allowed to rise freely. This prevents the loss of water caused by the previous re-set of water level to the ground. A damping routine is required to negotiate the rapid transition between DENOM values at this point.

The downstream sections of the trunks in these sewer systems are well below river levels and, therefore, we tried to use the tidal gate feature to model flap gates. The BOUND subroutine had a re-set of depth to the river level even though the gate was supposed to be closed. This caused the program to ignore the flap gates. We corrected this problem by inserting a statement to re-set the pipe depth of water to the correctly computed value. This modification has only been tested for gates located at the outfall node and to this extent it is a site specific modification.



REDUCTION OF AS(J) UNDER SURCHARGE
MODIFICATION TO CORRECT LOSS OF WATER PROBLEM AT FLOODING

FIG. 13

SUMMARY

In summary, the modifications we have made to our version of the program are, or can readily be made, compatible with any existing versions of the RUNOFF, EXTRAN, or W.R.E. TRANSPORT blocks. We have not radically altered the program's computational methods. Our changes are generally expansions and adaptations of the original authors' techniques to meet particular requirements we have encountered.

Such changes were required in greater part due to our use of the Programs out of the context for which they were intended, rather than any inherent short-comings of the programs themselves.

After we had developed an understanding of the hydraulic aspects of the SWMM program we have found it to be an excellent planning and design tool.

In conclusion, we have found the SWMM Users Group meetings to be an important forum for the exchange of ideas and appreciate this opportunity to make a contribution through this presentation.

ACKNOWLEDGEMENTS

The three projects; Leila North, Kildare Avenue and St. John's/Polson were all undertaken for the City of Winnipeg, Waterworks, Waste and Disposal Division, whose engineering personnel have actively encouraged the use of mathematical modelling technology in their projects.

We wish to acknowledge the encouragement and help extended by the U.S. Environmental Agency through Harry Torno, The University of Florida through Wayne Huber and Water Resources Engineers, Inc. through Larry Roesner.

SWMM USERS MEETING-MONTREAL, QUE.

MAY 24-25, 1979

SIMPLIFIED SEGMENTATION OF AN URBAN CATCHMENT FOR URBAN RUNOFF SIMULATION WITH SWMM

by

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1. INTRODUCTION

Numerous rainfall-runoff models have been developed and discussed in the last ten years to simulate urban runoff. Carefull thoughts should be given, in a drainage study, to the selection of a model. Ville de Laval, second largest city in the province of Québec, had to select a rainfall-runoff model for the revision of its drainage master plan. SWMM was selected tentively for several reasons, namely:

- the model is readily available
- the model benefits from an important technical back-up both at EPA and at Environment Canada
- the computer program of the model has a modular structure
- the model can simulate water quality and snowmelt
- the model, in its EXTRAN version can simulate specific flow conditions such as pressure flows.

Selecting SWMM for a predominantly planning study was surprising because it was not conceived for planning purposes and because SWMM uses a fairly large data bank.

Before final selection can be made, a study was conducted to assess if the data needed for SWMM were available and to determine a strategy for segmenting urban catchments.

1.1 Data bank

Available to the drainage analyst group was an up-to-date land use data bank, developed primarily for the revision of the water supply masterplan. The land use data bank provides information on the actual spatial distribution and forecasts future land use patterns for several date in the next thirty years. Land use is expressed in terms of residential, industrial and commercial areas, information that has to be transformed into physiographic parameters.

1.2 Simplified segmentation of urban catchment

A search for a successful strategy for simplified segmentation of an urban catchment was undertaken. Such a strategy is determinant in the selection of SWMM for planning purposes; firstly it allows the simulation of urban runoff at a lower computationnal cost than a detailed segmentation of the catchment; secondly it reduces the level of spatial resolution in the physiographic data needed to represent the urban catchment thus the size of the data bank.

Final selection of SWMM for the Laval study is conditionnal on the identification of a simplified segmentation procedure.

1.3 Articulation of the conference

In this conference the translation of the land use data bank into information for SWMM, transformation adopted for the revision study of the Ville de Laval drainage masterplan is first presented. This is followed by the description of the simplified segmentation strategy: comparisons between detailed and simplified hydrographs are illustrated.

2.0 LAND USE DATA BANK

Ville de Laval shares a common border line, the Rivière des Prairies with the Montréal Urban Community. Laval has a total area close to a hundred square miles (259 Km^2), the main drainage basin has an area of sixty square miles (155 Km^2). This basin is experiencing rapid urbanization namely in its southern region.

For land use purposes, the city is divided into 1200 zones each associated with a predominant type of land use; they are: residential, industrial, commercial, recreation area, transportation, institutional, agrucultural.

Residential zones are characterized by their population density (persons/acre), which has been normalized to describe the type of residence found in the zone, specifically single unit house, duplex, cottages, apartment buildings, row houses, etc...

2.1 The node data bank

For drainage purposes, the city is divided into subbasins of approximately 425 acres area (this number corresponds to the largest operationnal unit area of the simplified segmentation procedure). Each subbasin is associated to a node, a point of the future main drainage network. The ensemble

2.1 The node data bank (Cont'd)

of nodes constitutes the node bank; which contains the information used by SWMM as transformed from the land use information. Thus the node data bank is the interface between the land use data bank and the runoff model.

2.2 Estimation of runoff model parameters

Two parameters of the runoff process are directly obtained from land use: the percentage of impervious area and the drainage density. Both parameters strongly influence the overland flow runoff hydrograph.

The percentage of impervious area corresponding to a given land use, has been assessed through field investigations in Laval. Table 1 shows the percentage associated to each category of residential zones; table 2 presents this information for non-residential zones.

The drainage density is a function of the length of overland flow; it is essential to model correctly this variable as it influences significantly the peakedness of the simulated hydrograph. The width of overland flow (W) accounts in SWMM for drainage density and is estimated for each zone as the ratio of total area and mean depth of each lot. The latter in urban residential area is fairly uniform and in Laval was found to be about 130 feet; for other land use zones a model value was adopted, as shown in table 3.

TABLE 1

PERCENTAGE OF IMPERVIOUSNESS IN LAVAL'S RESIDENTIAL AREAS

TYPE	POPULATION DENSITY PERS/ACRE		DESCRIPTION	IMPERVIOUSNESS %
	From	To		
1	1	12	Single-family habitation with lots over 10,000 sq. feet of area	20
2	13	25	Single-family habitation with lots between 5000 and 10,000 sq. feet of area	38
3	26	40	Town houses, duplex, triplex adjoining single-family habitations	50
4	41	60	2 or 3 stories multi-family habitation	68
5	61	80	2 or 3 stories adjoining multi-family habitations	68
6	81	+	High-rise multi-family habitations	60

TABLE 2

PERCENTAGE OF IMPERVIOUSNESS IN AREA OF DIFFERENT LAND USES (NON-RESIDENTIAL AREAS)

TYPE OF LAND USE	DESCRIPTION	IMPERVIOUSNESS %
Industrial	Industries, manufactures, warehouses	80 to 95
Commercial	Commerces, offices buildings, hotels, motels, restaurants, service stations	95
Institution	Educational establishments, detention areas, churches, hospitals...	70
Parks	Parks, playing ground, golfs, wooded area, cemetery	7
Transport	Provincial large roads	55
Services	High tension electric transportation area, railways...	10
Agricultural	Agricultural area and other zones not urbanised	5

TABLE 3

MEAN DEPTH OF LOT (NON-RESIDENTIAL AREAS)

TYPE OF LAND USE	MEAN DEPTH (ft)
Industrial	120
Commercial	150
Institution	130
Parks	300
Transport	50
Services	100
Agricultural	300

2.3 Land use data to runoff parameters

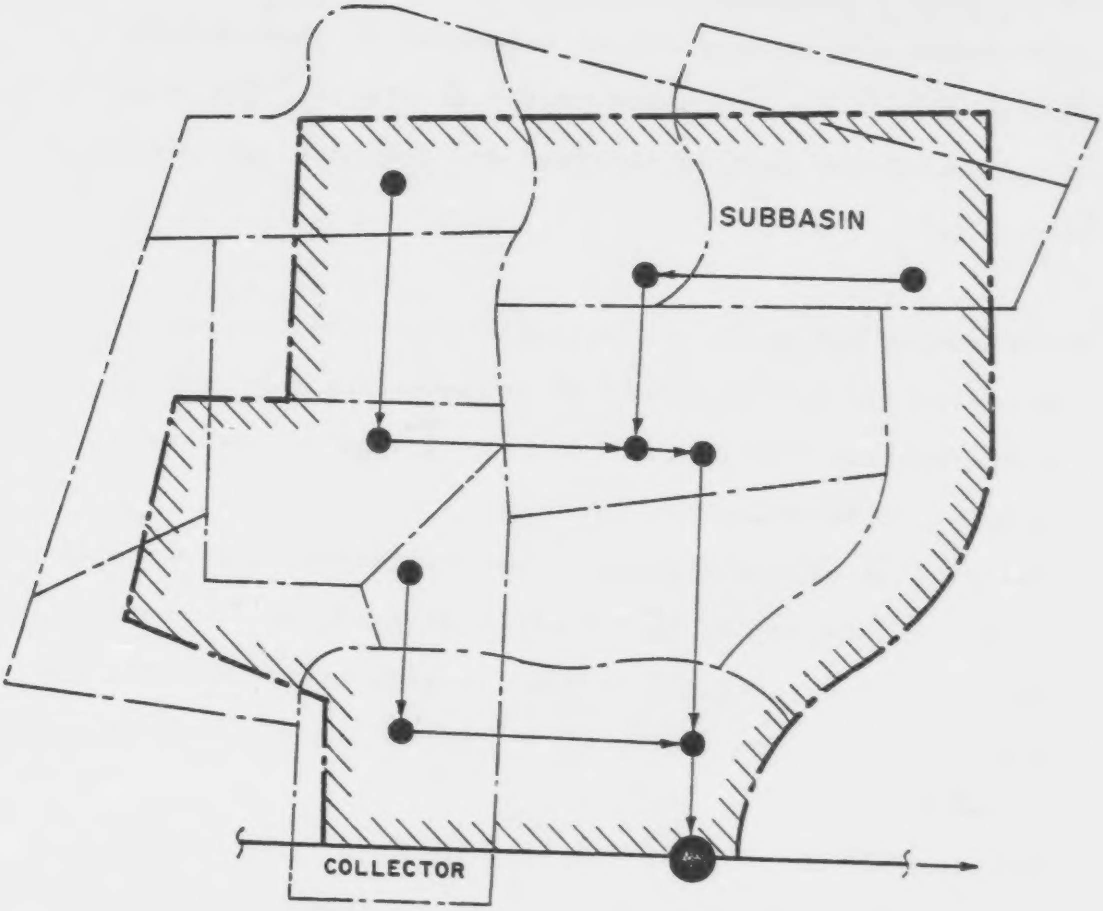
A computer model has been written (Desjardins & Sauriol (1977)) to estimate the parameters of the runoff model for each subbasin (each node of the node data bank) from the land use data bank.

The model identifies the zones or fraction of zones included in the subbasin. Figure 1 illustrates a typical case where some zones overlap one or more subbasins. It estimates the percentage of impervious areas in each subbasin as the mean of each zone, or fraction of, included in the subbasin and the width of overland flow as the sum of their respective W; the subbasin area is also computed from the land use data bank.

The model also computes the dry weather flow in each subbasin from its total estimated population, or its land use if not residential.

Other parameters, namely roughness, overland flow slope, and infiltration constants are presently assumed constant. Transport elements are not estimated from land use information but are defined from network on topographic maps.

FIGURE 1


SUBBASIN
COLLECTOR

TYPICAL LAND USE AND DRAINAGE BASIN OVERLAP

LEGEND

SUBBASIN BOUNDARY	— - —
LAND USE ZONE	— - —
PRIMARY NODE	⬤
SECONDARY NODE	●
TRANSPORT ELEMENT	——→

3.0 SIMPLIFIED SEGMENTATION OF AN URBAN CATCHMENT

3.1 Simplified segmentation criteria

A catchment may be modelled or segmented in various ways depending upon the objective of the simulation, the characteristics of the problem analyzed and the level of resolution sought.

For planning purposes, the segmentation criteria are:

- minimization of the number of segments while preserving the essential features at the hydrographs derived with a detailed segmentation
- description of the main drainage parameters which significantly influence the runoff, such as land use, topography, etc.
- description of internal (within the transport network) controls (quality and quantity) and of the main transport elements
- description of external(controls those modifying the inlet hydrograph). External controls may be structural or non structural.

A good strategy for simplified segmentation must yield results whose resolution is compatible with the objectives of the study; the results should also be reliable in order to select the drainage elements of the future network and the (quality, quantity) control alternatives.

3.2 Development of the segmentation procedure

A simplified segmentation contains as few drainage elements as needed to reproduce the basic runoff characteristics of the site.

Aggregating several small sub-catchments into a single one whose area is the sum of the subcatchment areas and whose parameters are averaged over these subcatchments is a priori interesting strategy for planning purposes, both from a technical and practical stand-points.

There were three phases in the development of the segmentation procedure. Firstly a simplified procedure was tested for the 23.3 acres Gray Haven catchment, an experimental basin of the Johns Hopkins University Storm Drainage Research Project, documented by Tecker (1969). Leclerc and Schaake (1973) for MITCAT (Harley (1967)), proposed a simplified segmentation for Gray Haven. Secondly the proposed procedure was applied to a 53.15 acre pilot basin in Laval to test it to local conditions. Thirdly a search was conducted to identify the largest size of a unit catchment compatible with the objectives of the study and the numerical capabilities of SWMM.

3.2 Development of the segmentation procedure (Con't)

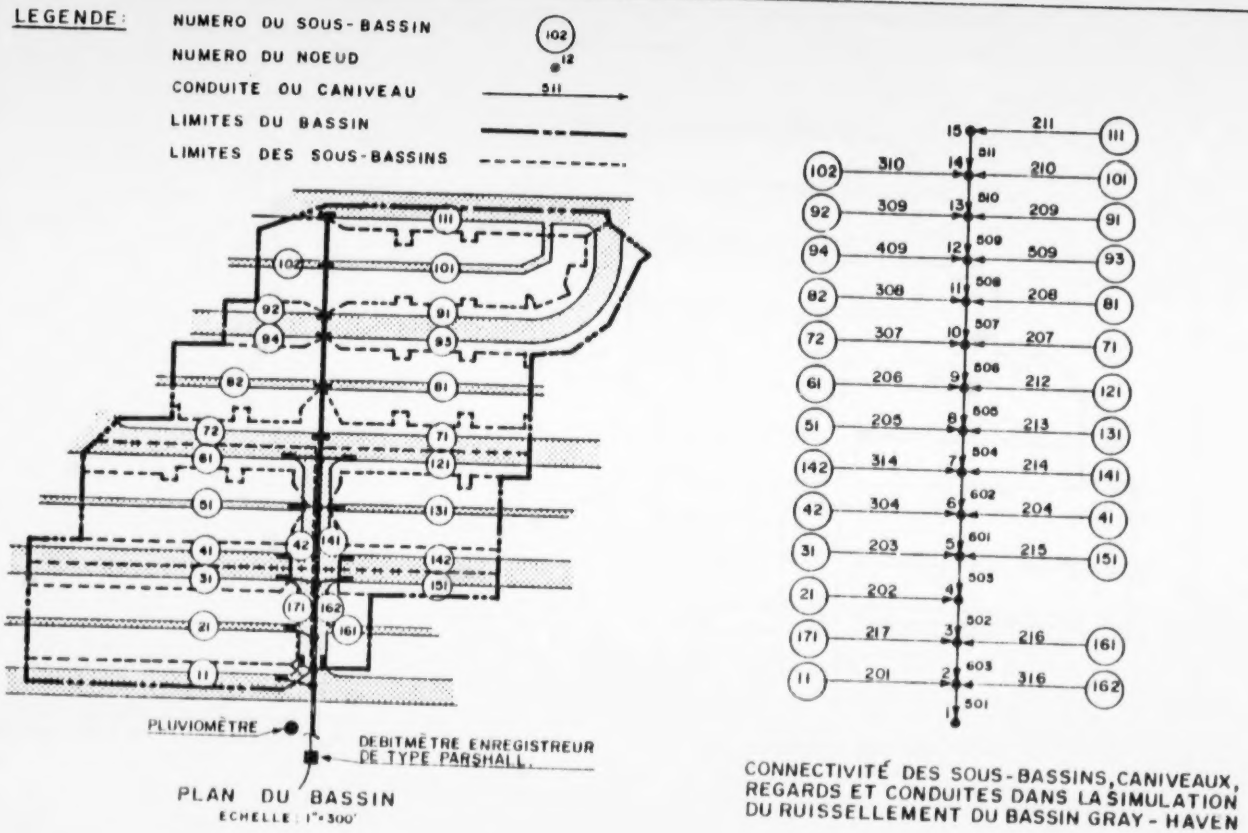
Gray Haven segmentation

Figures 2 and 3 show the detailed and simplified segmentations of the 23.3 acre catchment. The former is made of 26 subbasins, 26 gutters, 14 pipes and 14 nodes, the latter of one basin and one gutter whose length is equal to the longest transport distance. The runoff parameters in the simplified version is equal to the weighted areal average of the parameters of each subbasin, with the exception of the width of overland flow, W, which is equal to the summation of each variable.

Figure 4 illustrates the goodness of fit getween the detailed and simplified hydrographs.

53.15 acre Laval test basin

Using the procedure tested on Gray Haven, this test basin was represented by a single catchment and a single gutter Comparisons between simplified and detailed hydrographs showed that the procedure can be applied to a catchment of this size; this test pilot basin is refered to as the unit module.

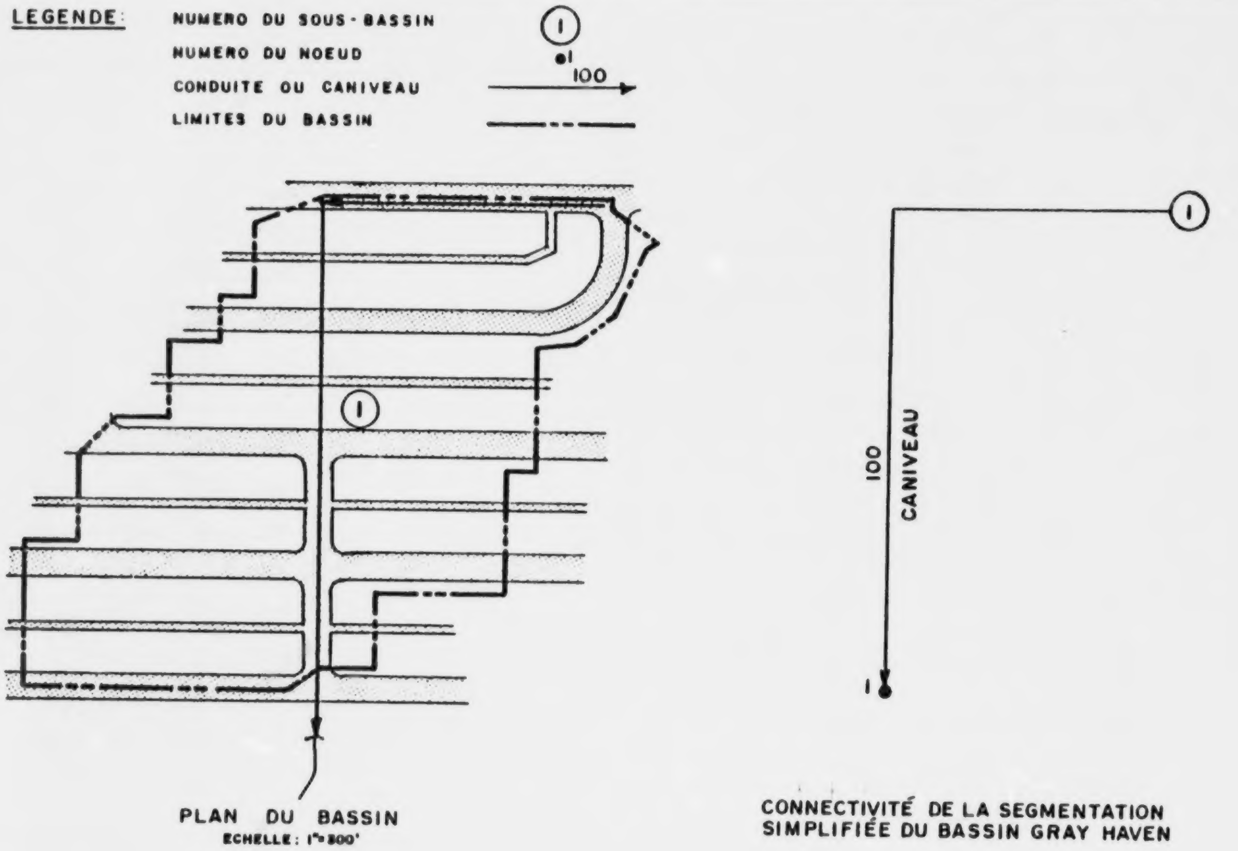

LEGENDE:
NUMERO DU SOUS-BASSIN
102
NUMERO DU NOEUD
12
CONDUITE OU CANIVEAU
511
LIMITES DU BASSIN
LIMITES DES SOUS-BASSINS
111
102
101
92
91
94
93
82
81
72
71
61
121
51
131
41
42
141
142
31
151
171
162
21
161
11
PLUVIOMÈTRE
DEBITMÈTRE ENREGISTREUR DE TYPE PARSHALL
15
211
14
511
310
210
13
510
309
209
12
509
409
509
11
508
308
208
10
507
307
207
9
506
206
212
8
505
205
213
7
504
314
214
6
602
304
204
5
601
203
215
4
503
202
3
502
217
216
2
603
201
316
1
501

PLAN DU BASSIN
ECHELLE: 1"=300'

CONNECTIVITÉ DES SOUS-BASSINS, CANIVEAUX, REGARDS ET CONDUITES DANS LA SIMULATION DU RUISSELLEMENT DU BASSIN GRAY-HAVEN

GRAY HAVEN DETAILED SEGMENTATION

FIGURE: 2

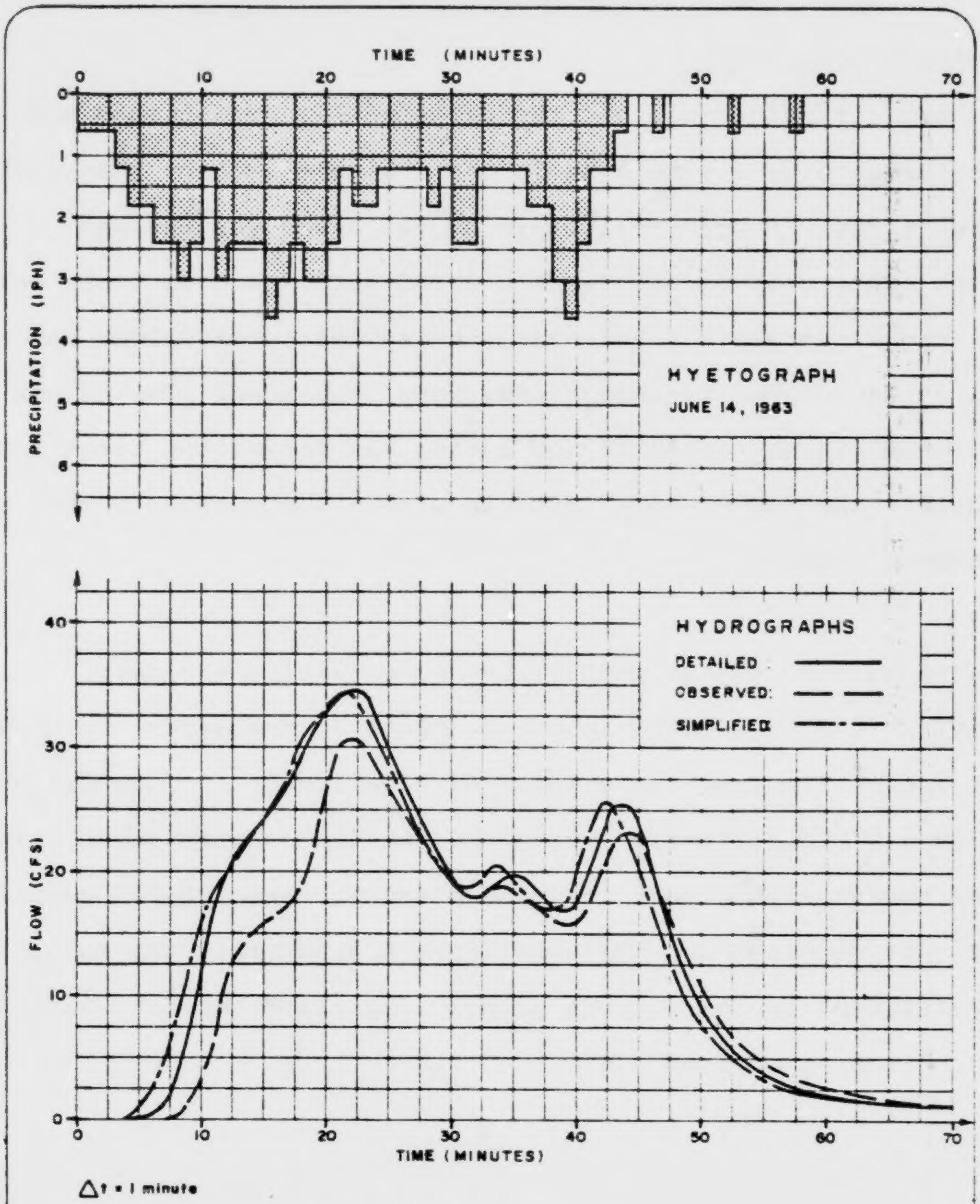

LEGENDE:
NUMERO DU SOUS-BASSIN
NUMERO DU NOEUD
CONDUITE OU CANIVEAU
LIMITES DU BASSIN
1
1
100
1
100
CANIVEAU
1
1

PLAN DU BASSIN
ECHELLE: 1"=300'

CONNECTIVITÉ DE LA SEGMENTATION
SIMPLIFIÉE DU BASSIN GRAY HAVEN

GRAY HAVEN SIMPLIFIED SEGMENTATION

FIGURE: 3



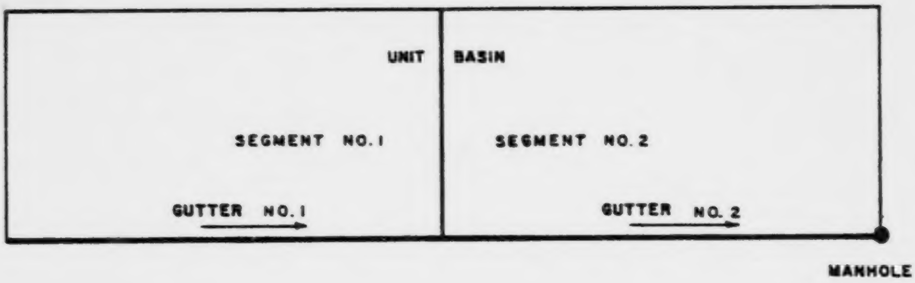
DETAILED, OBSERVED AND SIMPLIFIED HYDROGRAPHS, GRAY HAVEN, JUNE 14, 1963 FIGURE:4

3.2 Development of the segmentation procedure (Con't)

Largest unit basin

Pursuing the aggregation, two unit modules were lumped and represented by a single basin and a single gutter; then four, then eight. A practical limit of 425 acres was reached when eight unit modules were modeled with a single basin and a unique gutter; a non-convergence in the runoff block was experienced when intense storms were simulated. The volume of water to be routed in the gutter was indeed too large to satisfy the convergence criteria within the maximum number of CANSWMM version.

To avoid convergence problem the simplified segmentation adopted divides the unit basin into two identical subbasins, as shown in figure 5. They can either be in series or in parallel as this does not significantly modify the simulated hydrographs. It was however found that to improve the goodness of fit detailed (8 unit modules) and the simplified hydrographs, the Manning coefficient in the gutters of the simplified version, had to be increased from 0.013 to 0.018.


UNIT BASIN
SEGMENT NO. 1
SEGMENT NO. 2
GUTTER NO. 1
GUTTER NO. 2
MANHOLE

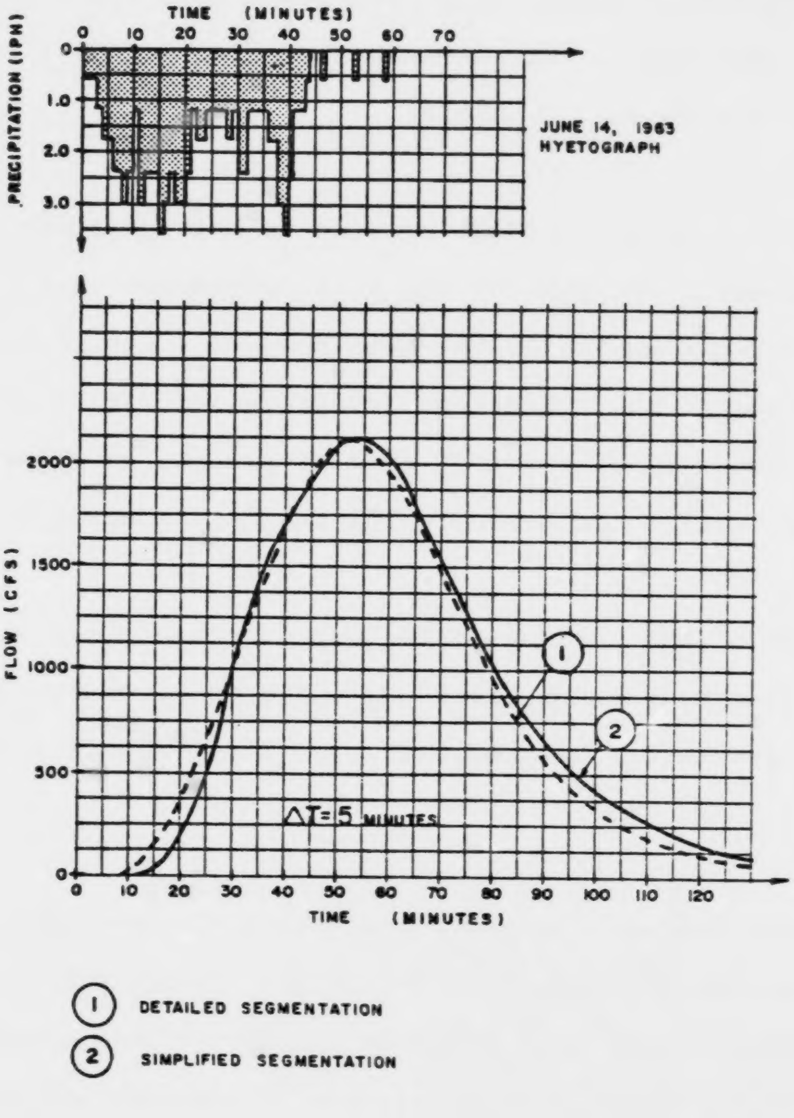
UNIT BASIN REPRESENTATION

FIGURE: 5

3.2 Development of the segmentation procedure (Con't)

Finally this simplified procedure was applied on the Vimont-Nord catchment, a 2225 acre basin in Laval. The simplified segmentation consisted of 6 subbasins while the semi-detailed version contained 23 subbasins. Figure 6 illustrates the goodness of fit of the resulting simulated hydrographs.

Based on these overall results, the procedure was adopted for the revision of the drainage masterplan of Laval.



VIMONT NORD DETAILED AND
SIMPLIFIED HYDROGRAPHS

FIGURE:6

4.0 DISCUSSION AND CONCLUSION

The simplified segmentation procedure identified for urban catchment in Laval makes the selection of SWMM for planning purposes, feasible.

Additional work should be devoted to this question namely a systematic analysis of the interdependance of Δt and the largest unit basin, of the magnitude of Δt with respect to the shape of the hydrograph (averaging effect of long Δt), and eventually of the influence of simplified segmentation upon simulation of urban runoff quality indicators.

The use of the land use data bank in conjunction with the node data bank makes assessment of a modification in the drainage network or the land use, upon urban runoff a relatively simple task.

The methodology adopted in Laval uses the available land use information efficiently and provides flexibility to plan the drainage network and the urban runoff control alternatives, at various stages of urbanization of the city of Laval.

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PRÉDICTION DE LA QUALITÉ DES EAUX DE RUISSELLEMENT UNE APPROCHE DIFFERENTE

WATER QUALITY PREDICTION FOR URBAN RUNOFF AN ALTERNATIVE APPROACH

By

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Colloque SWMM Users Group
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INTRODUCTION

The accurate prediction of storm water quality resulting from non-point sources of pollution has recently become a significant area of interest. In general, storm water quality is a function of the total pollutant accumulation on all surfaces exposed to rainfall. However, the rainfall intensity and wash-off capacity of the pollutants are also important factors. After pollutants have been dislodged from exposed surfaces, overland flow and channel hydraulics determine the pollutant concentrations evidenced at a downstream observation or sampling site. Recent studies[1] have indicated that pollution accumulation rates are affected by several factors including land use, population density, impervious area, traffic intensity, condition of surface area, total overland flow length time from last rain, street sweeping frequency, climate, and season.

A water quality prediciton strategy, as normally formulated, requires the compilation of a data base, calibration of a predictive model, simulation of existing and future conditions, analysis of these predictions, and use of the results in planning decisions[2]. Consequently, the data base provides a foundation on which both the analysis and subsequent decisions will be carried out. Because of the importance of valid data to the overall strategy, numerous investigators have attempted to compile data which could be used to develop predictive correlations between land use, watershed characteristics, runoff, and water quality. The result of these efforts is a very comprehensive data base for areas throughout the United States. Unfortunately, the data collection and analysis procedures utilized by each investigator were rarely uniform and consequently different sets of data have to be carefully processed to be comparable.

INITIAL DEVELOPMENTS

The original attempts to predict stormwater quality concentrated on regression analyses between watershed aspects and pollutant concentrations or loads[1,3,4]. The resulting equations were site specific and frequently not adaptable to different areas. A typical example of this type of equation is presented in Table 1.

The most extensively used deterministic model for predicting stormwater quality is represented by the following equation:

$$P_{t+\Delta t} = P_t e^{-KR\Delta t}$$

where:

P_t = Total pounds of pollutants on surface of the watershed at the begining of the time interval

t = the measure of time from the start of the storm

Δt = the time interval over which the integration is occuring

K = the decay constant or rate of pollutant washoff

R = the actual runoff rate measured in inches per hour

This equation was originally developed as part of the EPA Storm Water Management Model (SWMM) and has been subsequently introduced into numerous other water quality predictive capability models including the U. S. Army Corps

TABLE 1

EQUATIONS OF BEST FIT FOR THE THIRTEEN WATER QUALITY CONSTITUENTS*

1) Suspended Solids, mg/ℓ =
    $21.55 + 4.36 \text{ LOG } (Q/A)$ for $Q/A \leq 0.75$
    Suspended Solids, mg/ℓ =
    $37.83 + 134.7 \text{ LOG } (Q/A)$ for $Q/A > 0.75$
2) Dissolved Solids, mg/ℓ = $155.02 - 40.25 \text{ LOG } (Q/A)$
3) Ammonia, mg/ℓ = $0.465 - 0.078 \text{ LOG } (Q/A)$
4) Organic Nitrogen, mg/ℓ = $0.306 + 0.071 \text{ LOG } (Q/A)$
5) Nitrates, mg/ℓ = $0.188 + 0.148 \text{ LOG } (Q/A)$
6) Total Phosphorus, mg/ℓ =
    $0.0366 - 0.957 \text{ LOG } (Q/A)$ for $Q/A \leq 0.305$
    Total Phosphorus, mg/ℓ =
    $0.508 - 0.042 \text{ LOG } (Q/A)$ for $Q/A > 0.305$
7) BOD, mg/ℓ = $4.11 - 0.282 \text{ LOG } (Q/A)$
8) COD, mg/ℓ = $34.43 + 10.12 \text{ LOG } (Q/A)$ for $Q/A \leq 5.6$
    COD, mg/ℓ = $46.32 - 5.77 \text{ LOG } (Q/A)$ for $Q/A > 5.6$
9) Fecal Streptococci, 1000 counts/100 ml =
    $1010\ (Q/A)^{3.24}$ for $Q/A \leq 0.22$
    Fecal Streptococci, 1000 counts/100 ml =
    $15.35\ (Q/A)^{0.472}$ for $Q/A > 0.22$
10) Total Coliform, 1000 counts/100 ml = $17.4\ (\text{Fecal Strep})^{1.463}$
11) Fecal Coliform, 1000 counts/100 ml =
    $0.152\ (\text{Total Coliform})^{0.767}$
12) Total Insecticides, μg/ℓ = $0.269 + 0.11 \text{ LOG } (Q/A)$
13) Total Herbicides, μg/ℓ = $0.158 + 0.038 \text{ LOG } (Q/A)$

---

*All logarithms are base 10.

of Engineers Storage Treatment Overflow Runoff Model (STORM) and the Illinois State Water Survey QUAL - ILLUDAS Model. In application, this equation is reliable if the initial concentration of pollutants and the washoff rate is adequately known[5]. One approach to evaluating the initial concentration is to use a literature value, most notably from an American Public Works Association Study of the Chicago Area[6]. The values from that work are based on surface samples taken from various streets in and around the south side of Chicago. Because only street surfaces were sampled, the data neglects pollutant accumulation on non-street areas. In general, experience has shown the Chicago values rarely give accurate results in SWMM modeling, however, they are still used when estimating the relative storm water quality impacts in areas where little or no recorded quality data exists.

Another method for estimating the initial concentration is by calibration against recorded storm water quality data. This method requires estimating the initial concentration and generating a pollutograph for an actual rainfall and runoff event. The computed pollutograph is then compared to a recorded pollutograph. If the two vary significantly, the initial concentration is revised and a new pollutograph generated. This trial and error procedure is continued until a satisfactory version has been achieved. Unfortunately these results are only site specific and therefore not easily transferable.

In view of the previous discussion, any alternative approach would have to be generally applicable in at least a regional scope, as well as, easily determined from a minimum data collection and analysis effort. Consequently the new method, which is the subject of this presentation, was developed.

DATA SELECTION AND ANALYSIS

Ten watersheds from 7 different regions of the United States were chosen for analysis. The data for a total of 112 runoff events was compiled. Table 2 presents brief descriptions of the selected watersheds. The analysis of the data base was specifically directed towards the development of a predictive model which, with given parameters of runoff quantity and watershed characteristics, will predict storm water quality.

To compare the quantity - quality relationships between different watersheds, and hence different surface and land use characteristics being studied, it was necessary to reduce the data to a common format. Because each watershed is different, in terms of the drainage area, ground slope, soil type, and vegetation, it is obvious that different runoff volume and pollutant quantities would be produced even if other factors such as land use and rainfall were equal. For this reason, runoff volumes and pounds of pollutants were converted to inches of runoff and average pounds of pollutant per acre generated during each storm event.

A previous study[7] had identified significant functional relationships between pollutant loads and runoff volume. The two quantity-quality relationships which showed reasonable linear association were logarithmic transforms of:

1. Total runoff volume in inches versus the total unit load in pounds per acre.
2. Cumulative runoff volume in cubic feet versus the cumulative pollutant load in pounds.

TABLE 2
DESCRIPTIONS OF WATERSHEDS USED FOR ANALYSIS

Watershed	Area (ac.)	Percent Impervious	Number of Events	Description of Land Use
1. 39th and Holdrege Lincoln, Neb.	79.	24.	18	Older family dwellings and a limited number of multi-family dwellings. Assume 90% S.F.R. and 10% M.F.R.
2. 63rd and Holdrege Lincoln, Neb.	85.	22.5*	14	10 thru 15 year old single family residential. Assume 100% S.F.R.
3. 78th and A Lincoln, Neb.	375.	15.0*	10	Area under development for single family residential and consisting of homes less than 10 years old, homes under construction, undeveloped lots and farm land.
4. Third Fork Durham, N.C.	1069.	29.0	35	Consists of 59% residential development; 19% commercial and industrial; 12% public and institutional lands; and 10% vacant.
5. Panther Branch Woodlands, Tex.	16050.	1.0*	4	Primarily undeveloped woodlands.
6. Panther Branch Woodlands, Tex.	21007.	10.0*	5	Area under development for single and multi-family residential with related commercial developments.
7. Hunting Bayou Houston, Tex.	1976.	23.0	4	Residential and commercial with some industrial developments.
8. K.N. Clapp Basin Lubbock, Tex.	223.	30.0	11	Essentially residential development with no industrial development, although one commercial shipping center with accompanying parking lot and two schools are present.
9. Selby Street San Francisco, Ca.	3400.	22.0	7	Consists of 77% residential; 2% commercial; 6% industrial; and 15% vacant land.
10. Alameda Floodway, Albq., N.M.	65920	23.0*	4	Residential and commercial with undeveloped upper reaches.

*Estimated based on land-use.

The reliability of these relationships was statistically verified by use of the correlation coefficient and its standard deviation.

For all storm events and pollutants examined, a reasonably straight line relationship was found to exist. This relationship was found to hold true on both the total and cumulative runoff versus pollutant relationships. As a result of this analysis, it was decided that the total load runoff relationship would be used as a basis for more detailed statistical studies.

ALTERNATE METHOD DEVELOPMENT

The correlation analysis results indicated that the logarithmic transforms of total runoff volume in inches and total pollutant load in pounds per acre yielded the highest degree of linear correlation with correlation coefficients ranging from 0.77 to 1.00 and having a median value of 0.93. Therefore, best fit equations were generated for total suspended solids, biochemical oxygen demand, chemical oxygen demand, Kjeldahl-nitrogen, ammonia nitrogen, total phosphates and dissolved oxygen for each watershed. This analysis consisted of first transforming the variables, total runoff in inches, and total pounds per acre, to their logarithmic values and then performing a linear regression on the variables. The second correlation between logarithmic transforms of cumulative pounds of pollutant and cumulative runoff in cubic feet also proved to be successful. Sample results are presented in Figures 1, 2, and 3.

Both correlation equations are of the form:

$$Y = ax^b$$

where:

Y = total pounds of pollutant per acre or cumulative pounds at time, t

x = total runoff in inches or cumulative runoff in cubic feet at time, t

a, b = regression coefficients

Table 3 presents the generated regression coefficients "a" and "b" for the total volume and load correlation for the ten study watersheds. It is postulated that in the case of of Area 10, Alameda Floodway, Albuquerque, New Mexico, the definition of runoff volume in inches may be inaccurate because of the non-uniform rainfall patterns typical for this area. Higher rainfall volumes would elevate these curves and perhaps reduce the extreme differences shown here.

To evaluate the statistical significance of the regression analysis, a test utilizing the standard deviation of the correlation coefficients (s_r) was performed. The results of this analysis indicate a generally higher degree of significance, with standard deviations ranging from 0.17 to 1.00 and a median value of 0.85.

In attempting to relate these regression equations to different surface characteristics found on each watershed, a number of previously identified parameters were examined. The only parameter which appeared to produce rational association with runoff quantity and quality was percent imperviousness of each watershed. In general, as the percent imperviousness of the watershed increases, the pollutant load generated by a specific runoff event was found to increase. This relationship is illustrated in Figure 4, for total suspended solids. Therefore, using percent imperviousness, a statistical analysis was conducted in an attempt to relate this parameter to the quantity-quality equations previously developed. The results of this analysis indicate a strong correlation between variations in the imperviousness of a basin and the previously defined coefficient "a". Coefficient "b", on the other hand, exhibits no such correlation, and seems to cluster around a value approaching 1.0. From this analysis, a final set of total runoff quantity and total quality functions is presented in Table 4. Given the percent

FIGURE 1

TOTAL POLLUTANT LOADINGS AS A FUNCTION OF RUNOFF

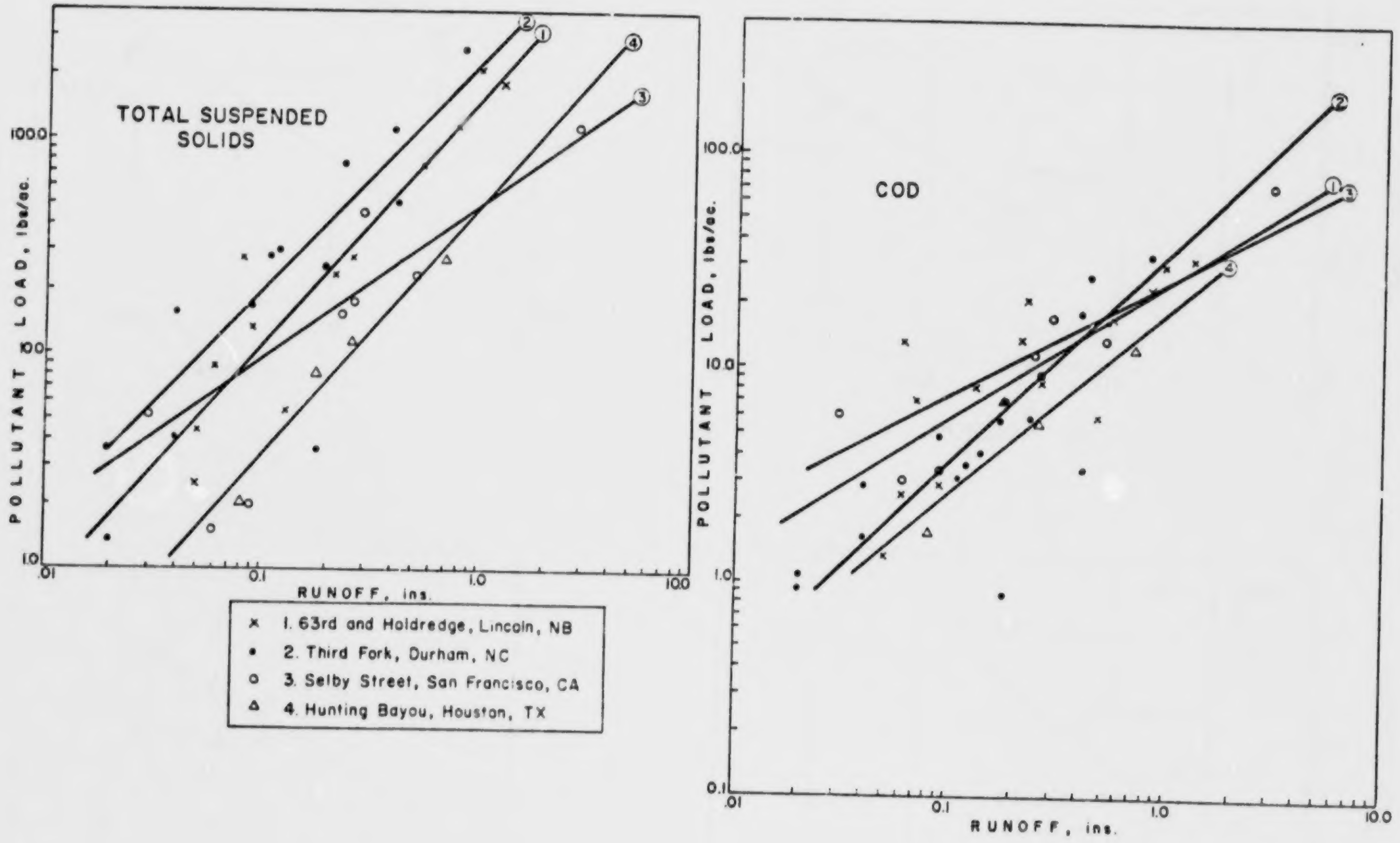


FIGURE 2
WATER QUALITY RELATIONSHIP - COD

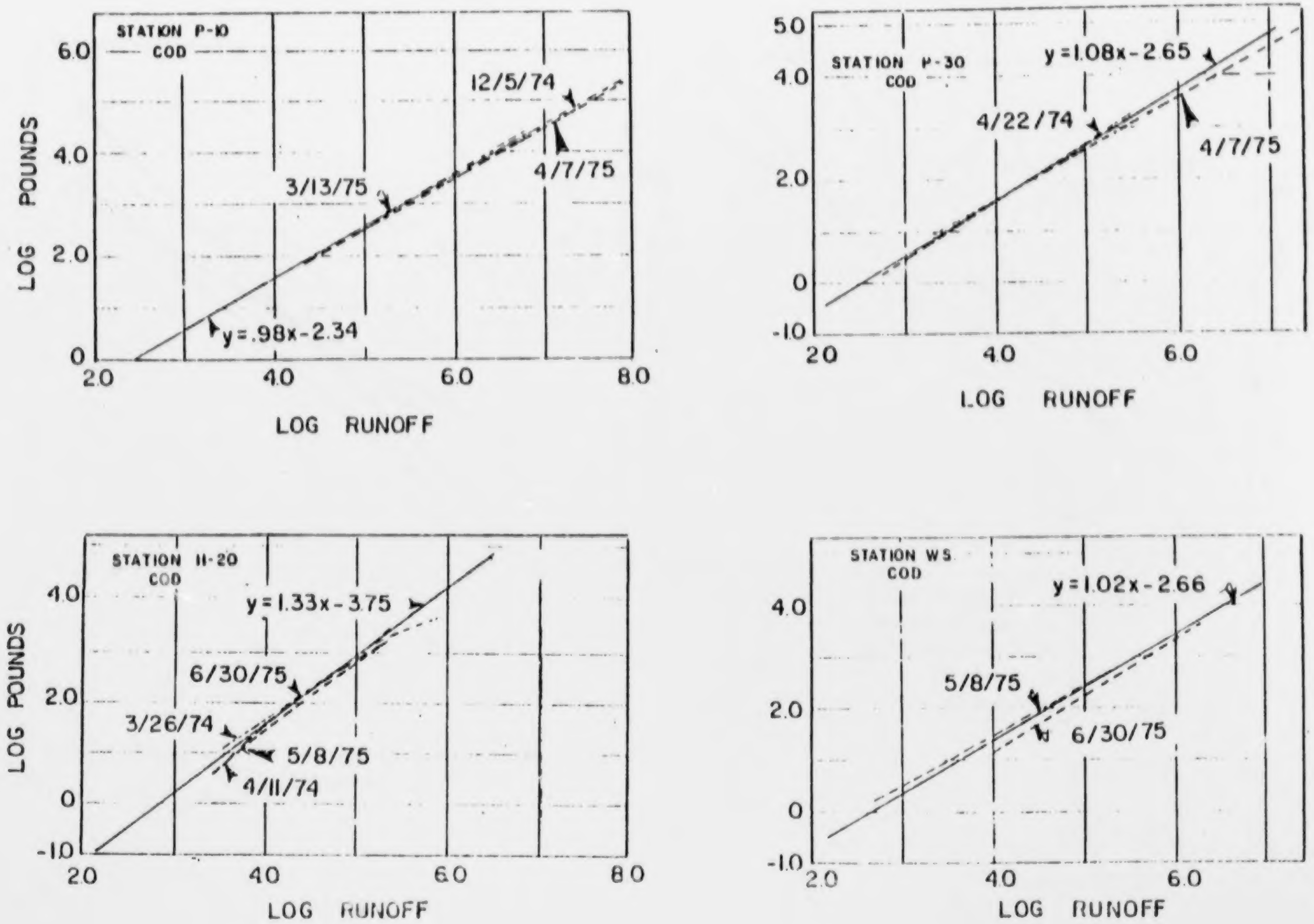


FIGURE 3

WATER QUALITY RELATIONSHIP - SUSPENDED SOLIDS

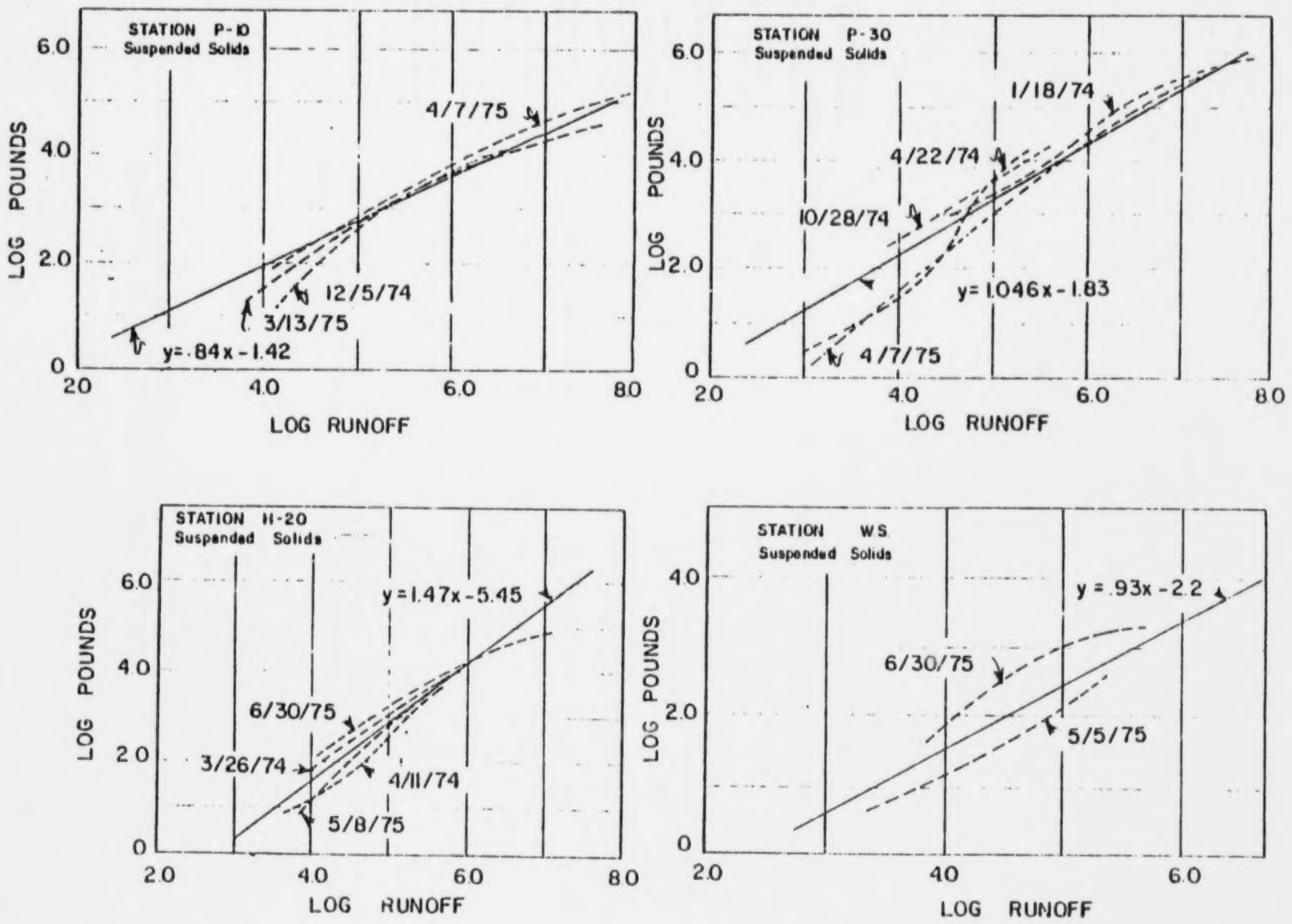


TABLE 3

SUMMARY OF REGRESSION COEFFICIENTS DERIVED FROM THE QUANTITY/QUALITY ANALYSES

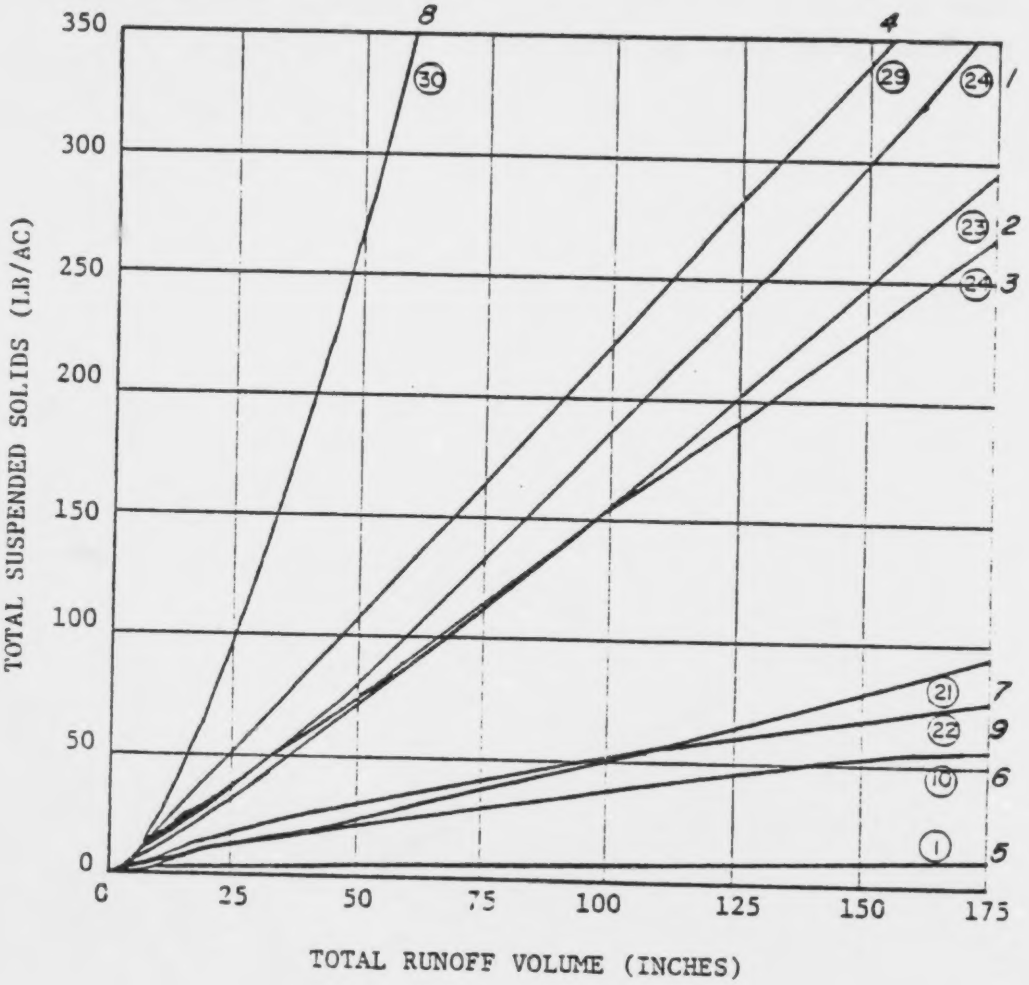
WATERSHED	NUMBER OF EVENTS	TSS		B.O.D.(5)		C.O.D.*		KJELDAHL-N		NH_3-N		TOTAL PO_4	
		a	b	a	b	a	b	a	b	a	b	a	b*
1. 39th and Holdrege Lincoln, Neb.	18	185.88	1.19	3.69	.65	38.81	.34	---	---	---	---	---	---
2. 63rd and Holdrege Lincoln, Neb.	14	156.26	1.14	2.89	.35	26.24	.64	---	---	---	---	---	---
3. 78th and A Lincoln, Neb.	10	155.75	.97	1.08	.71	15.46	.95	---	---	---	---	---	---
4. Third Fork Durham, N.C.	35	222.66	1.05	13.15	1.18	35.78	.99	.08	.73	---	---	.19	1.08
5. Panther Branch Woodlands, Tex.	4	7.16	.75	---	---	18.51	1.14	.25	.92	.02	1.01	.02	.99
6. Panther Branch Woodlands, Tex.	5	38.94	.77	---	---	12.49	.89	.39	.99	.02	.77	.02	.67
7. Hunting Bayou Houston, Tex.	4	49.77	1.16	---	---	19.85	.87	1.17	1.39	.79	1.63	.2	1.02
8. K.N. Clapp Basin Lubbock, Tex.	11	763.63	1.54	27.65	1.70	---	---	---	---	---	---	.06	1.41
9. Selby Street San Francisco, Ca.	7	51.19	.72	4.77	.31	26.52	.54	.63	.47	.12	.38	.11	.19
10. Alameda Floodway Albuquerque, N.M.	4	---	---	---	---	14454	2.56	32.4	1.94	234.4	2.98	30.9	1.94

of imperviousness of a basin and the runoff volume in inches, this function can be used to generate total pollutant load in pounds per acre for a given runoff event.

The correlation between watershed parameters and the coefficients of the cumulative runoff-water quality equations has not been sufficiently investigated at this time to define acceptable modifications as those conducted for the total load relationships.

FIGURE 4

CHANGE IN THE QUANTITY/QUALITY OF
STORM WATER RUNOFF AND PERCENT IMPERVIOUS



㉒ Percent Imperviousness of Watershed
9 Watershed Number

TABLE 4

SUMMARY OF POLLUTANT YIELD (LB/AC) QUANTITY/QUALITY EQUATIONS

GENERAL EQUATION: $\text{Load} = a * x^b$

1) TSS

$\text{lbs TSS/acre} = I/(.1362 + .00384(I)\ (\text{inches runoff})^{1.05}$

2) BOD(5)

$\text{lbs BOD(5)/acre} = 7.3971\ (83.0334/I)\ (\text{inches runoff})^{.82}$

3) COD

$\text{lbs COD/acre} = 12.71e^{.0315(I)}\ (\text{inches runoff})^{.86}$

4) KJELDAHL-N

$\text{lbs KJD-N/acre} = .01369e^{.0888(I)}\ (\text{inches runoff})^{.90}$

5) NH_3-N

$\text{lbs } NH_3\text{-N/acre} = .2437e^{.0438(I)}\ (\text{inches runoff})^{.95}$

6) Total Phosphates

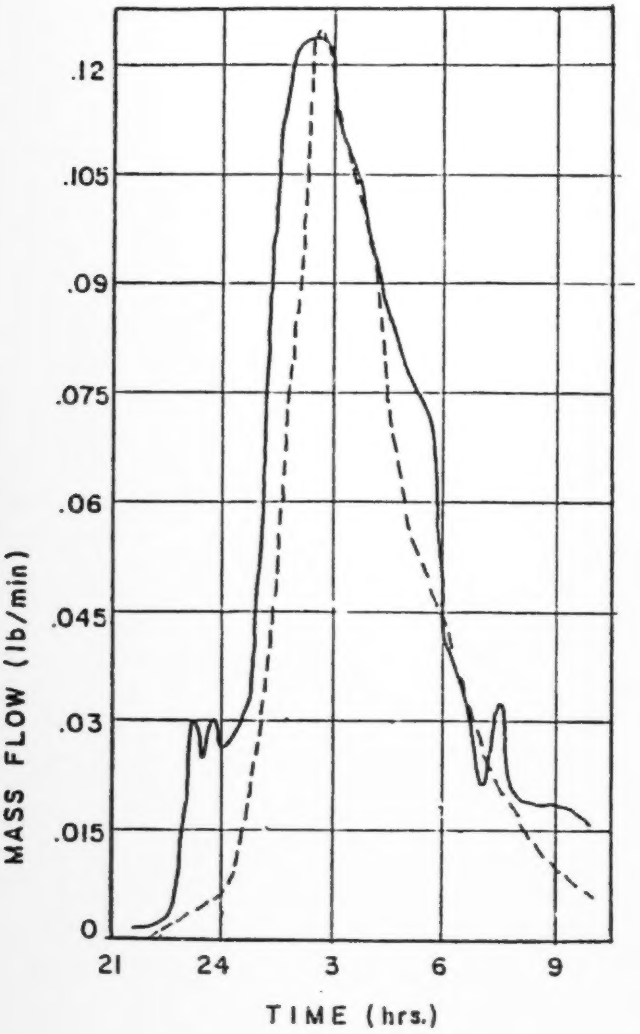
$\text{lbs Total Phosphates/acre} = .09677(I)\ (\text{inches runoff})^{.89}$

APPLICATION OF ALTERNATE METHOD

This method can be very simply applied to determine both pollutographs and total loads from a given rainfall runoff event, using the following procedure:

1. Initially, runoff quantity and rate will have to be determined by other hydrologic methods generally available (Rational Equation, Unit Hydrograph, Kinematic Wave Routing, etc.).
2. The hydrograph generated for either a design storm or an observed storm can then be converted to a mass flow accumulation curve.
3. The data from this curve can be used in conjunction with the cumulative runoff versus cumulative pollutant load curves (Figures 2 and 3) and the corresponding pollutant load accumulations may be determined. At present, the site specific or comparable data is required to determine the pollutant load accumulations; however, as more data become available and the regionalization of the data is verified, dependency upon data collection will be reduced.
4. The pollutant load accumulations may then be converted to pollutant concentrations by means of the flow volume and the imcremental pollutant loads. The result of this exercise will be a pollutograph an example of which is presented in Figure 5.

FIGURE 5
MASS TRANSPORT OF NITRATES AT STATION H-20
STORM OF 5/08/75, HUNTING BAYOU



5. If only total pollutant loads are desired, or to verify the previously computed pollutant loads, the alternate set of correlations between total pollutant load and total runoff may be used.

FUTURE DIRECTIONS

The relatively simple and facile method to predict stormwater quality as presented herein should prove most effective in determing total pollutant loads and associated pollutographs in most streams and rivers. The linear relationships between runoff and pollutant transport indicate a direct proportionality between pollutant load and urbanization as measured by percent imperviousness of the area.

This storm water quality prediction method has received only limited testing and verification. Furthermore only the total load and pollutant relationships have been correlated to impervious area on a multi-regional basis. A similar analysis has to be conducted for the accumulative runoff and pollutant relationships which currently are only available for specific regions.

Also, as more data become available in other areas not previously considered, the correlations developed thus far must be re-evaluated and verified or revised as neccessary. However, major revisions are not anticipated.

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WASTEWATER INTERCEPTION ON THE COMMUNAUTE URBAINE DE MONTREAL TERRITORY

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Translated from original text in french
by Denis Burroughs, P.E.

WASTEWATER INTERCEPTION ON THE COMMUNAUTE URBAINE DE MONTREAL (CUM) TERRITORY

1- INTRODUCTION

In early 1975, the "Communauté urbaine de Montréal" started the construction of a system of interceptors as it engaged itself in a vast program for the treatment of its wastewaters. The twenty-nine (29) municipalities which form the CUM regroup approximately 1,900,000 inhabitants on a territory that totals about 123,000 acres.

Presently, the developped portion of the CUM territory amounts to about 90,000 acres, 75% of which are drained by a combined sewer system, while only 25% of the area is served by a separate sewer network.

In the ultimate stage of development, 63% of the area will be drained via a combined sewer system, and 37% via the separate or pseudo-separate sewer system.

The different areas of the CUM served by combined, separate or pseudo-separate sewer systems are shown on figure 1, which also indicates the location of existent and proposed interceptors.

With the exception of the Ste-Hélène and Notre-Dame islands (which have their own treatment plants) and the industrial sector of the eastern portion of the island (whose wastewaters will be treated locally), the North and South Interceptors will concentrate the wastewater generated by the CUM's 120,000 acre drainage area, towards one single and gigantic wastewater treatment plant located at the eastern tip of the island.

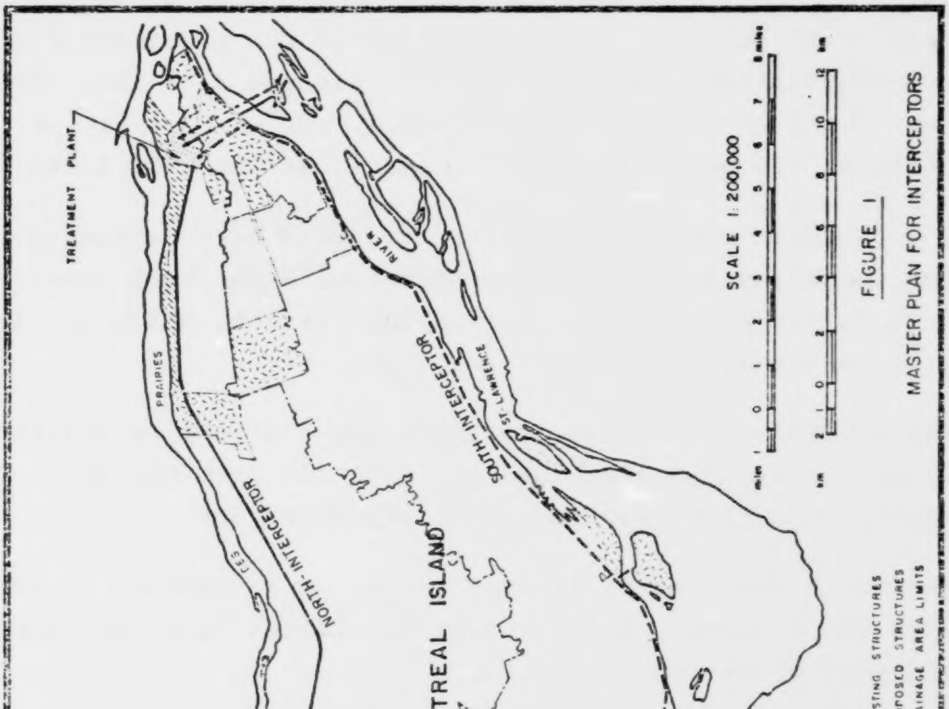

TREATMENT PLANT
RIVIÈRE DES PRAIRIES
NORTH-INTERCEPTOR
MONTREAL ISLAND
SOUTH-INTERCEPTOR
ST-LAWRENCE RIVER
LAKE SAINT-LOUIS
SCALE 1: 200,000
miles 1 0 1 2 3 4 5 6 7 8 miles
km 2 0 2 4 6 8 10 12 km
LÉGEND
SEPARATE SEWER
PSEUDO SEPARATE SEWER
COMBINED SEWER
TREATMENT PLANT LOCATION
INDUSTRIAL AREA NOT DRAINED TO TREATMENT PLANT
AREA SERVED BY NOTRE DAME TREATMENT PLANT
EXISTING STRUCTURES
PROPOSED STRUCTURES
DRAINAGE AREA LIMITS

FIGURE 1

MASTER PLAN FOR INTERCEPTORS

The existing portion of the North Interceptor, with an approximate length of 120,000 ft., was rock-tunneled at depths varying from 50 to 100 ft. At the entrance of the treatment plant's pumping station, the invert of the sixteen (16) foot diameter horseshoe shape interceptor is located at the -50' elevation (M.S.L.), while at its exit, the water level is at the +49' level mark.

The proposed South Interceptor will also be constructed using the tunneling method. Its diameter will be about nineteen (19) feet at the entrance of the pumping station, while its invert will be set at about -100' (M.S.L.).

The maximum flow at the treatment plant's pumping station is evaluated at 3520 cfs, that is, 1560 cfs from the North Interceptor, and 1960 cfs from the South Interceptor.

The North Interceptor is expected to be in service in 1982 when the treatment plant's pumping station is to be completed and made operational.

For the time being, the construction schedule of the South Interceptor has not yet been finalized.

The cost of this CUM project, including both interceptors and the sewage treatment plant, is estimated at 1.2 billion dollars.

2- DESIGN CRITERIA FOR THE NORTH INTERCEPTOR

2.1 Mean Dry-weather flows (M.D.W.F.)

The values chosen for design are given in Table 1. These values were derived from a water consumption study and a flow measurement program carried out over the CUM's entire territory.

TABLE 1

NORTH INTERCEPTOR - MEAN DRY WEATHER FLOW (M.D.W.F.) CALCULATIONS

Land Use	Unit flow	Area (acres)	Ultimate mean dry weather flow (cfs)
Residential	75 Imp. gal/day - person	30689	180.2
Industrial	0.008 cfs/acre (sector 1) 0.012 cfs/acre (sectors 2 &3)	11656	132.2
Commercial	0.007 cfs/acre (sector 1) 0.009 cfs/acre (sectors 2 & 3)	2227	18.6
Institutional	0.007 cfs/acre (sector 1) 0.010 cfs/acre (sectors 2 & 3)	3024	28.4
Parks	-	5129	-
Transport	-	915	-
Infiltration	0.0006 to 0.015 cfs/acre for 1972 developped areas 0.0016 to 0.004 cfs/acre for 1972 non-developed areas	53640	199.0
			558.4

* Ultimate population: 1,297,839 inhabitants

2.2 Maximum flows

The total maximum flow carried by the interceptors is equal to the sum of the maximum flow from the sanitary sewer system (separate) and the maximum intercepted flow from the combined or pseudo-separate sewer system.

The maximum flow from the different sanitary sewer drainage areas is function of the average flow cumulated along the length of the interceptor. It can be computed using the following equation derived from the CUM's flow measurement program.

$$Q_{max} = 6.40\, Q_{mean}^{\,0.856} \qquad \text{(cfs)}$$

The maximum flow intercepted from a combined or pseudo-separate sewer drainage area is equal to the mean flow of the drainage area multiplied by one of the following factors:

- 4.0 x M.D.W.F. for all drainage basins served by pseudo-separate systems, as well as for all the combined-sewer basins located west of the Laurentian Autoroute, plus the small basins which will not be equipped with automatic control regulators.
- 2.5 x M.D.W.F. for all combined-sewer basins which are connected to the interceptor east of the Laurentian Autoroute, and equipped with automatic control regulators (permitting maximum use of the interceptor's capacity).

The M.D.W.F. multiple of 4.0, chosen for the small combined-sewer basins and the pseudo-separate basins, corresponds approximately to the average peak factor for dry weather flows observed throughout the CUM for drainage basins of these dimensions. This value was thus chosen to preclude any overflow during dry-weather periods.

This value of 4.0 was also adopted for the remaining combined sewer drainage areas west of the Laurentian Autoroute to insure adequate protection of the existing water intakes of Ville de Laval, located on Rivière des Prairies.

The intercepted flow for the larger combined sewer drainage areas equipped with automatic control regulation devices is 2.5 times the M.D.W.F. This value should prevent any overflows during the dry-weather periods and especially during the spring thaw, when the flow reaches a value of 2 and 3 times the Mean Dry Weather Flow (M.D.W.F.). This M.D.W.F. multiple also considerably reduces the volume and number of overflow events and their duration during wet weather periods.

Table 2 shows the influence of the ratio of intercepted flow to M.D.W.F. on the characteristics of the overflows.

However, due to the nature of regulator control adopted by the CUM, to the vastness of the territory covered, and to the off-setting of inlet flows to the interceptor, the number and importance of the overflows should be considerably less than those expected on a single collector.

A computer simulation study showed that there was no need to further increase the M.D.W.F. multiple for combined-sewer drainage areas. As illustrated in Table 3, any increase in this value has but a negligeable effect on the volume of the overflow during wet weather.

It is to be clearly noted that these simulations were made considering a uniform storm over the entire CUM territory. For isolated storms however, due to the use of automatic control regulators, the volume of overflow to the river would be considerably less than that shown in Table 3.

TABLE 2

OVERFLOW CHARACTERISTICS OF THE MEILLEUR-ATLANTIQUE COLLECTOR AS A FUNCTION OF INTERCEPTION CAPACITY

Interception capacity as "n" times M.D.W.F.	Overflow duration		number of events	% of wastewater as overflow
	hours	% time		
2	514	5.8	157	6.0
2.5	265	3.0	82	4.2
3	163	1.9	62	3.1
4	87	1.0	38	2.0
5	39	0.4	26	1.4
6	25	0.3	19	1.0

Note: Study made with the measured flow of the Meilleur-Atlantique collector for the december '67 to december '68 period. (developped area: 4600 acres)

TABLE 3

VOLUME OF OVERFLOWS FOR THE ULTIMATE STAGE OF DEVELOPMENT

(million cubic feet)

Storm Frequency	With regulator control				Without regulator control			
	Interception capacity as "n" times M.D.W.F.				Interception capacity as "n" times M.D.W.F.			
	1.5	2.0	2.5	3.0	1.5	2.0	2.5	3.0
4/year	1.8	1.2	0.8	0.5	5.9	5.0	4.1	3.5
2/year	7.8	6.5	5.6	4.8	14.3	13.3	12.1	11.1
1/year	18.4	15.9	13.9	12.5	25.5	24.4	22.8	21.6
1/2 years	37.0	34.1	31.5	29.2	44.6	43.3	41.3	39.9
1/5 years	52.2	49.2	46.4	44.0	60.0	58.7	56.4	55.6
1/10 years	77.5	74.2	71.3	68.6	85.8	84.9	81.7	80.0

Note: Simulations made considering a sanitary flow of 1.0 x M.D.W.F.

3- DESIGN OF INTERCEPTION STRUCTURES

Presently, nearly all of the sanitary collectors are discharged directly, via pumping stations, to the waters surrounding the island, without any treatment. When the interceptors will be in operation, the sanitary sewer collectors will be connected directly to the drop-structures of the interceptor and the present pumping stations will be abandoned.

Every combined sewer collector will be connected to the interceptor via a regulation structure that will ensure maximum utilization of the interceptor capacity.

Figure 2 depicts the method of regulation that is to be adopted.

This derivation structure is essentially a chamber built below the invert of the existing or proposed combined sewer collector. The dimensions of the chamber were chosen to deviate a certain portion of the flow of the collector towards the interceptor via a pipe whose crown is located slightly below the invert of the collector.

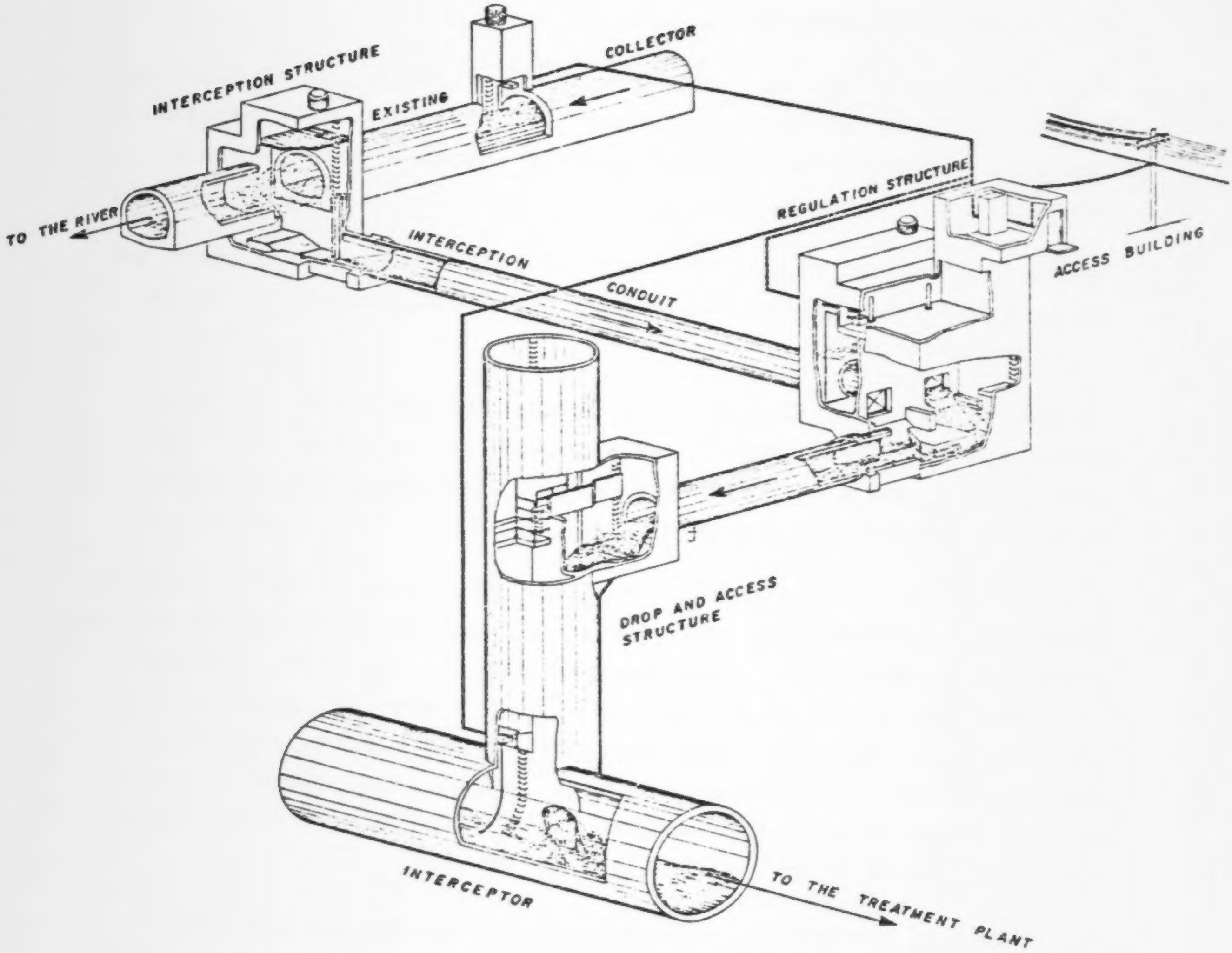
Sluice gates, in the regulation structure, control the flow to the interceptor. They are actuated by the command signals coming from the central console located at the treatment plant or from the local controller. Water then flows into the drop well to the interceptor and towards the treatment plant.

To ensure maximum utilization of interceptor capacity, when available, the regulator structures are individually dimensioned to allow, on the whole, more flow into the interceptors than the interception capacity designed for, particularly to cope with isolated or localized storms.

To obtain a certain uniformity in the design and in the construction details of the interception structures, their dimensions were established, after scale studies undertaken at the Laboratoire d'Hydraulique LaSalle, for regulator capacities of 150, 300, 450, 600 and 750 cfs.

FIGURE 2

OPERATING SCHEME FOR THE REGULATION STRUCTURES


INTERCEPTION STRUCTURE
EXISTING
COLLECTOR
TO THE RIVER
INTERCEPTION
CONDUIT
REGULATION STRUCTURE
ACCESS BUILDING
DROP AND ACCESS
STRUCTURE
INTERCEPTOR
TO THE TREATMENT PLANT

The nominal capacity for each interception structure was chosen with regard to the theoretical flows for different storm frequencies the theoretical capacity of the collectors, the capacity of the interceptor at the point of junction with the collector and finally the receiving water quality to be maintained at the point of overflow.

The storm which can be intercepted at a given point of junction, if the capacity of the interceptor is available, will generally fluctuate from a frequency of four (4) times a year to one of once in two (2) years.

Table 4 gives the characteristics of the existing regulators or those to be constructed before the North Interceptor is to be made operational.

It should be noted that all regulators designed with hydraulic actuators and the Curotte regulator will be equipped with an auxiliary power plant to assure their continuous functioning even during power failures.

A study by Dr. Paul Edouard Brunelle, ing., from l'Université de Sherbrooke, showed that even when the interceptor flows at maximum capacity at the plant's pumping station, in the case of a power failure, the resuming of service with the use of 2 pumps of 223 cfs each, equipped with auxiliary power plants, ensures adequate control of the system with no excessive surge of water in the North Interceptor, assuming the gates of the regulation chambers can be closed within 2 to 3 minutes.

The O'Brien, St-Laurent, 25e Avenue and 54e Avenue regulators will not be equipped with auxiliary power plants because of their temporary nature: their collectors are to be rebuilt eventually.

4- CONTROL SYSTEM FOR THE REGULATOR STRUCTURES

General description of the control system

TABLE 4

CHARACTERISTICS OF REGULATORS TO BE IN OPERATION WHEN THE NORTH INTERCEPTOR GOES INTO SERVICE

Regulator identification	Area drained (acres)	Collector capacity (cfs)	Ultimate M.D.W.F. (cfs)	Regulator Capacity (cfs)	Number and size of gates	Actuator type
Pitfield **	926	proposed	8.9	200	2-4'x 4'	hydraulic
Autoroute des Laurentides *	9655	2624	119.6	600	2-6'x 6'	hydraulic
O'Brien **	1823	335	20.0	150	1-4'x 4'	electric
Meilleur **	3455	2234	46.5	600	2-6'x 6'	hydraulic
St-Laurent **	816	251	10.6	150	1-5'x 5'	electric
Auteuil **	1643	955	23.5	300	2-4'x 4'	hydraulic
Curotte **	2883	1096	43.4	300	2-4'x 4'	electric
Lauzanne *	2005	600	25.6	300	2-4'x 4'	hydraulic
Hénault *	631	400	8.3	300	2-4'x 4'	hydraulic
Salk *	116	150	1.2	150	1-4'x 4'	hydraulic
Lanthier *	362	400	4.9	150	1-4'x 4'	hydraulic
Langelier *	1781	950	22.7	450	2-5'x 5'	hydraulic
25e avenue *	798	560	9.2	150	1-5'x 5'	electric
54e avenue *	1107	850	14.3	150	1-5'x 5'	electric

* existing ** proposed 1982

The goals to be achieved by the control system advocated by the CUM are as follows:

- to minimize the storm overflows with the optimal use of the interceptor's capacity and the choice of interception sites as function of time and wastewater quality.

- to control the flow in the interceptor to ensure the optimal functioning of the pumping station.

- to avoid any backwater flow in the sewer system directly connected to the interceptor and especially in the case of a power failure at the pumping station.

The supervision and operation of this control system requires the acquisition and processing of a large quantity of data. Table 5 gives the list of the different types of measures and points of control needed for each regulator of the hydraulic type. To these must be added measures of the water level in the interceptor at different points of junction with the sanitary sewer collectors, water level in the river, surveillance of pumping stations and of the rain gauge network.

A schematic representation of the mode of operation to be provided for the regulating structures' control system is presented in figure 3.

A "remote intelligent multiplexer" located at each regulator structure will ensure the regulation-system's operation, and the transmission by telephone lines of the different measurement and control signals, to the central console located at the treatment plant.

The process computer in the treatment plant will maintain supervision over the whole control system of the North

TABLE 5

MEASURE AND CONTROL ITEMS FOR HYDRAULIC TYPE REGULATORS

Digital Measures

1. Hydraulic Actuators
    - cylinder low-pressure
    - reservoir-oil high-temperature
    - pump filter high differential pressure
    - master feeding pump high negative pressure
    - start-stop of auxiliary feeding pump engine
    - oil low-level in reservoir
    - actuator control mode
2. Generator
    - start-stop of generator
    - fuel low-level in reservoir
    - battery charge
    - state of power supply
3. Miscellaneous
    - local power failure
    - start-stop of ventilation system
    - presence of noxious gases in regulation chamber
    - gas detection system's failure
    - heating system malfunction
    - heat detector
    - presence of water in regulation chamber
    - no entry in building
    - manual flushing of bubbler tube
    - instrumentation air low-pressure
    - back water gate at the submerged outfall

Analog measures

- water level in interceptor
- water level in collector
- water level upstream of gates
- positioning of gates
- hydraulic fluid pressure in actuators
- actuators' hydraulic fluid temperature

Commands

- gates' commands
- start-stop of generator (emergency power supply)
- command of flushing of bubbler tube

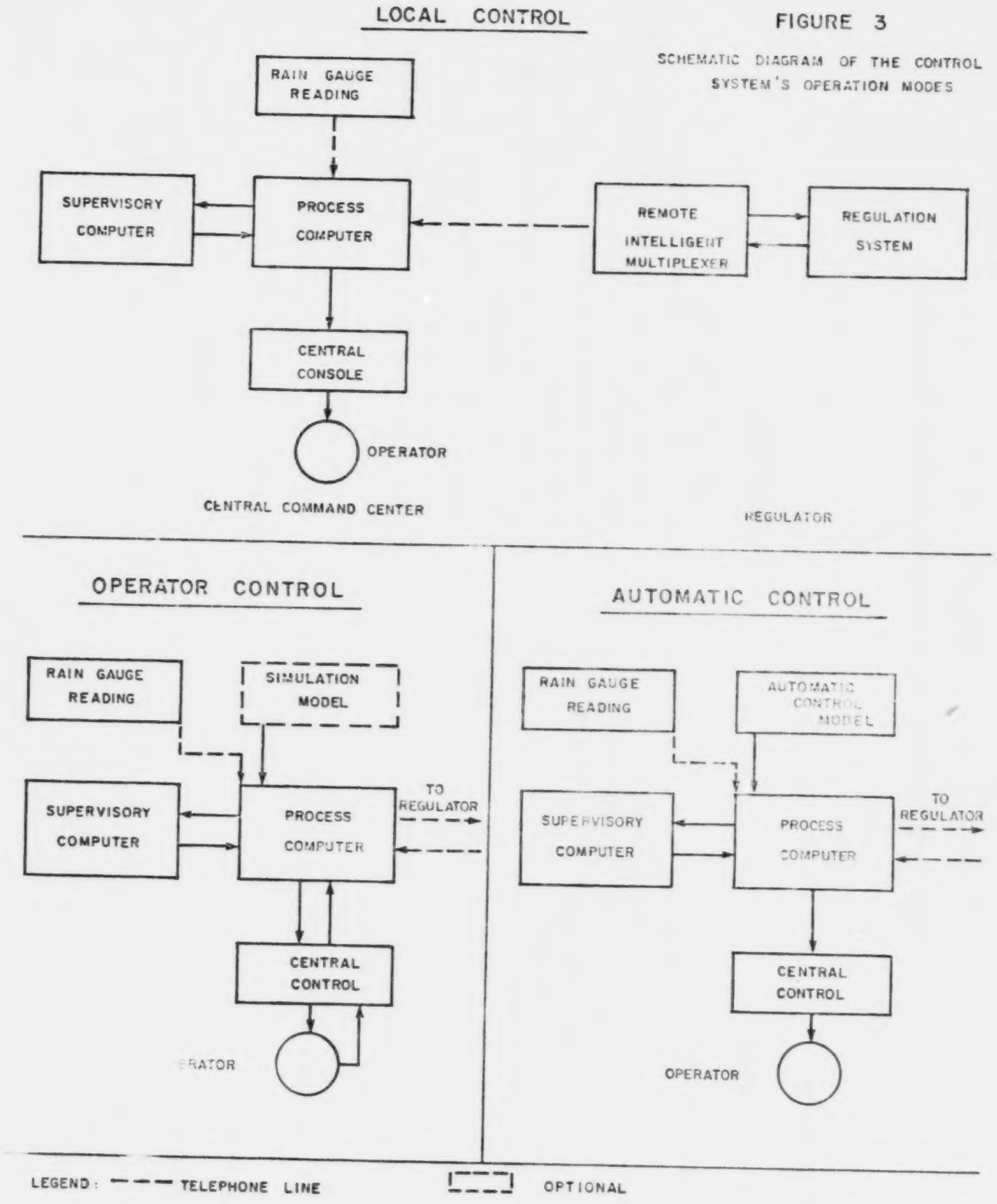


FIGURE 3

SCHEMATIC DIAGRAM OF THE CONTROL SYSTEM'S OPERATION MODES

LOCAL CONTROL
RAIN GAUGE READING
SUPERVISORY COMPUTER
PROCESS COMPUTER
REMOTE INTELLIGENT MULTIPLEXER
REGULATION SYSTEM
CENTRAL CONSOLE
OPERATOR
CENTRAL COMMAND CENTER
REGULATOR
OPERATOR CONTROL
RAIN GAUGE READING
SIMULATION MODEL
SUPERVISORY COMPUTER
PROCESS COMPUTER
TO REGULATOR
CENTRAL CONTROL
ERATOR
AUTOMATIC CONTROL
RAIN GAUGE READING
AUTOMATIC CONTROL MODEL
SUPERVISORY COMPUTER
PROCESS COMPUTER
TO REGULATOR
CENTRAL CONTROL
OPERATOR
LEGEND: TELEPHONE LINE
OPTIONAL

Interceptor regulators and will be able if required to carry out the execution operations depending on the mode chosen. A similar system will be provided for the South Interceptor regulators.

The master supervisory computer will assure the surveillance of all the different process computers, and will carry out memory banking of data, preparation of periodical reports, maintenance schedules, etc.

4.2 Local control of regulators

The first stage of regulator control that will be implemented after the interceptor is made operational will be the automatic local control of the regulators. The multiplexer will be programmed so as to maintain the gate positioning as a function of the water level in the collector and in the interceptor to ensure that the permissible water level in the interceptor is not exceeded.

Eventually, to obviate any difficulties stemming from failure of the central console, of the computer, of the communication system, or any other emergency situation, when the interceptor is functioning in another mode of control the commands will automatically be returned to the regulators local control to avoid forming a pressure head in the interceptor.

4.3 Control of regulators by the operator

Once the central console and the computer are installed, and the computer program controlling the opening and closing of the gates prepared, the operator will be able to control the regulators by giving the appropriate instructions to the process computer.

Evidently, in the beginning of the operations, this action will be very limited due to the restricted knowledge of the system. But after a certain breaking-in period, with the acquisition of data collected during the operation of the interceptor, a better knowledge of the behavior of the system will aid in the preparation of a simulation model which will facilitate the operator's work.

The model is to anticipate the flow to be intercepted at each derivation structure, the overflow to the river and the water level in the interceptor. With these informations, the operator will then be able to control the gates as a function of the flow admissible to the interceptor. A subsequent verification will be made to compare the effective water levels to the anticipated levels, and corrective actions will be taken if necessary.

4.4 Automatic control of regulators

The simulation model will serve as a basis for the preparation of the automatic control and operation model.

The automatic control model incorporates in its calculations, the real values of the water levels in the collectors, interceptors and regulation chambers. These calculations will permit the adjustment to real conditions in the interceptor of the values forecasted by the simulation model, to obtain finally an automatic real-time control of the whole system.

The process computer will then be able to operate the gates depending on the intercepted flow value submitted by the automatic control model.

Different studies now in progress at the CUM and l'Ecole Polytechnique de Montréal should help set up this control and operation program during the coming months.

4.5 Raingauge network

To optimize this wastewater interception system, there has to be a real time reading of the rainfall data so as to anticipate the flows in the collectors.

The CUM will eventually install fifty (50) raingauges that will transmit their data by telephone lines to the central console. Ten (10) tipping bucket raingauges are already installed on the roofs of different buildings throughout the CUM and are now operating and transmitting data to the Service d'assainissement des eaux. The data thus collected is presently processed and stored in the computer and will help in the study of storm characteristics in the Montreal Island Area and in the formulation of precise strategies for the regulation system's control.

5- CONCLUSION

With the help of a control and supervision system based on the use of computers located at the treatment plant, the CUM will be able to assure continuous monitoring and control of the interceptors, regulation structures and the treatment plant pumping station.

The control system advocated will also help to minimize storm overflows by assuring optimal use of interceptor capacity and by the choice of interception sites depending on wastewater quality to be intercepted and the priorities of the overflow points as a function of water use.

The work undertaken by the CUM will greatly help in improving the quality of the Montreal Island surrounding waterways.

PROCEDURE FOR THE ESTABLISHMENT OF STATEWIDE WASTELOAD ALLOCATIONS

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Presented at
SWMM User's Group Meeting
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INTRODUCTION

State regulatory agencies are faced with the problem of developing effluent limitations for industrial, semi-public and municipal dischargers which assure that water quality standards are met and that the economic burden of environmental protection is shared as equitably as possible among all dischargers to a stream segment. In the past, emphasis has been placed on the development of sound technical water quality modeling procedures and the calibration and verification of the water quality models.

The purpose of this paper is to present the minimum treatment requirements which have been established for dischargers by the United States Environmental Protection Agency and a procedure which allows regulatory agencies to decide where limited resources for water quality sampling and field investigation should be placed in developing defensible waste load allocations.

INDUSTRIAL POINT SOURCE ALLOCATION PROCEDURES

Waste load allocation analysis procedures for industrial dischargers must be developed in accordance with the Clean Water Act of 1977, Public Law 95-217. This law establishes revised criteria for the determination of effluent limitations for industrial dischargers. Under the Clean Water Act, pollutants discharged by industrial point sources are divided into three broad classifications: conventional pollutants, toxic pollutants, and nonconventional pollutants. The act defines conventional pollutants as biological oxygen demanding substances, suspended solids, fecal coliforms, and pH. All industrial treatment works are required to implement best conventional pollutant control technology (BCT) for conventional pollutants by July 1, 1984. Effluent limitations in excess of BCT are required in situations where BCT controls will result in continued violations of water quality standards.

Effluent limitations for toxic substances are determined by the implementation of best available technology economically achievable (BAT). Two separate lists of toxic substances with different implementation schedules are specified by the Act. All toxic pollutants referred to in Table 1 of Committee Print Number 95-30 of the Committee on Public Works and Transportation of the House of Representatives must meet BAT effluent limitations by July 1, 1984. This list is presented in Table 1 of this report. Any pollutants added to this list by the Administrator of the U.S. Environmental Protection Agency require the implementation of BAT controls no later than three years after the time these pollutants are added to the list. Any pollutants which are not considered to be conventional pollutants or which do not appear on the toxic substances list, are referred to as nonconventional pollutants and are required to meet BAT effluent limitations within three years of the time limitations are established, or July 1, 1984 whichever is later.

The Environmental Protection Agency is currently conducting a review of BAT effluent limitations for those industries where BAT effluent limitations have previously been established, to determine what changes are necessary to conform to the BCT criteria. Table 2 lists the schedule which EPA has established to publish BAT effluent guidelines for toxic substances in individual industries. Any industries which currently lack BAT guidelines for conventional pollutants will receive BCT guidelines according to the same schedule.

TABLE 1: TOXIC POLLUTANTS LIST PURSUANT TO THE CLEAN WATER ACT OF 1977 (PUBLIC LAW 95-217)

1. Acenaphthene.
2. Acrolein.
3. Acrylonitrile.
4. Aldrin/Dieldrin.[1]
5. Antimony and compounds.[2]
6. Arsenic and compounds.
7. Asbestos.
8. Benzene.
9. Benzidine.[1]
10. Beryllium and compounds.
11. Cadmium and compounds.
12. Carbon tetrachloride.
13. Chlordane (technical mixture and metabolities).
14. Chlorinated benzenes (other than dichlorobenzenes).
15. Chlorinated ethanes (including 1,2-dichloroethane, 1,1,1-trichloroethane, and hexachloroethane).
16. Chloroalkyl ethers (chloromethyl, chloroethyl, and mixed ethers).
17. Chlorinated naphthalene.
18. Chlorinated phenols (other than those listed elsewhere; includes trichlorophenols and chlorinated cresols).
19. Chloroform.
20. 2-chlorophenol.
21. Chromium and compounds.
22. Copper and compounds.
23. Cyanides.
24. DDT and metabolites.[1]
25. Dichlorobenzenes (1,2-,1,3-, and 1,4-dichlorobenzenes).
26. Dichlorobenzidine.
27. Dichloroethylenes (1,1-, and 1,2-dichloroethylene).
28. 2,4-dichlorophenol.
29. Dichloropropane and dichloropropene.
30. 2,4-dimethylphenol.
31. Dinitrotoluene.
32. Diphenylhydrazine.
33. Endosulfan and metabolites.
34. Endrin and metabolites.[1]
35. Ethylbenzene.
36. Fluoranthene.
37. Haloethers (other than those listed elsewhere; includes chlorophenylphenyl ethers, bromophenylphenyl ether, bis (dichloroisopropyl) ether, bis-(chloroethoxy) methane and polychlorinated diphenyl ethers).
38. Halomethanes (other than those listed elsewhere; includes methylene chloride methylchloride, methylbromide, bromoform, dichlorobromomethane, trichlorofluoromethane, dichlorodifluromethane).
39. Heptachlor and metabolites.
40. Hexachlorobutadiene.
41. Hexachlorocyclohexane (all isomers).
42. Hexachlorocyclopentadiene.
43. Isophorone.
44. Lead and compounds.
45. Mercury and compounds.
46. Naphthalene.
47. Nickel and compounds.
48. Nitrobenzene.
49. Nitrophenols (including 2,4-dinitrophenol, dinitrocresol).
50. Nitrosamines.
51. Pentachlorophenol
52. Phenol.
53. Phthalate esters.
54. Polychlorinated biphenyls (PCBs).[1]
55. Polynuclear aromatic hydrocarbons (including benzanthracenes, benzopyrenes, benzofluoranthene, chrysenes, dibenzanthracenes, and indenopyrenes).
56. Selenium and compounds.
57. Silver and compounds.
58. 2,3,7,8-tetrachlorodibenzo-p-dioxin (TCDD).
59. Tetrachloroethylene.
60. Thallium and compounds.
61. Toluene.
62. Toxaphene.[1]
63. Trichloroethylene.
64. Vinyl chloride.
65. Zinc and compounds.

---

[1] Effluent standard promulgated (40 CFR Part 129).

[2] The term "compounds" shall include organic and inorganic compounds.

TABLE 2: PROPOSED DATES FOR PROMULGATION OF BAT EFFLUENT LIMITATIONS

Group 1 (September 30, 1978 (proposed), March 31, 1979 (final))

- Timber Products
- Steam Electric Power Plants
- Leather Tanning & Finishing
- Iron & Steel Manufacturing
- Petroleum Refining

Group 2 (December 31, 1979 (proposed), June 30, 1979 (final))

- Nonferrous Metals Manufacturing
- Paving & Roofing Materials (Tars and Asphalt)
- Paint & Ink Formulation and Printing
- Ore Mining & Dressing
- Coal Mining

Group 3 (March 31, 1979 (proposed), September 30, 1979 (final))

- Organic Chemicals Manufacturing
- Inorganic Chemicals Manufacturing
- Textile Mills
- Plastics & Synthetic Materials Manufacturing
- Pulp & Paperboard Products
- Rubber Processing

Group 4 (June 30, 1979 (proposed), December 31, 1979 (final))

- Soaps & Detergent Manufacturing
- Auto & Other Laundries (Industrial Laundries)
- Miscellaneous Chemicals
  - Pesticide Manufacturing
  - Photographic Products
  - Gums & Woods (Glue Making)
  - Pharmaceuticals
  - Explosives
  - Adhesives & Sealants
- Machinery & Mechanical Products Manufacturing
  - Battery Manufacturing
  - Plastics Manufacturing
  - Foundries
  - Coil Coatings
  - Porcelain & Enameling
  - Aluminum
  - Copper
  - Electronics
  - Shipbuilding
  - Mechanical Products (Metal Fabrication)
- Electroplating

In some cases, BAT guidelines will be published for nonconventional pollutants at the same time that BAT guidelines are published for toxic pollutants. In other cases, BAT guidelines for nonconventional pollutants will be postponed until a later time. However, as was the case when BPT guidelines were published, water quality standards must be met, even if effluent limitations more stringent than the appropriate BCT or BAT effluent limitations are required.

Until the BCT and BAT guidelines are issued, state regulatory agencies are faced with the problem of issuing NPDES permits for industries whose existing permits expire during 1978 or 1979. For those industries for which BCT or BAT guidelines have not been published, it is recommended that waste load allocations be developed based on meeting in-stream water quality standards. As a minimum, all industrial dischargers should be required to comply with BPT (current permit) effluent limitations. For those parameters for which no water quality standards have been established, and for which BAT or BCT guidelines have not been published, waste load allocation analysis should not be attempted.

MUNICIPAL AND SEMI-PUBLIC POINT SOURCE ALLOCATION PROCEDURES

Municipal and semi-public dischargers are required to implement, as a minimum, secondary treatment for all discharges. Where more stringent effluent limitations are required to insure compliance with water quality standards, we have recommended that discrete treatment levels be utilized, similar to those presented in Table 3. These treatment levels, listed in order of increasing stringency, reflect current wastewater treatment technology, are applicable throughout the planning period (i.e., 20 years), and are capable of being achieved with common unit process combinations. The use of specific treatment

TABLE 3: RECOMMENDED TREATMENT LEVELS FOR MUNICIPAL AND SEMI-PUBLIC DISCHARGERS

Treatment Removal Requirements			Effluent Concentration (Thirty Day Mean)				
Effluent Quality	BOD (% Removal)	Nitrification	BOD_5 (mg/l)	NH_3-N (mg/l)	Dissolved Oxygen (mg/l)	Effluent Reaeration	UOD ** (mg/l)
A	85-90	No	30	15	4.0	No	113
B	85-90	No	30	15	6.0	Yes	113
C*	85-90	No	20	15	4.0	No	98
D*	85-90	No	20	15	6.0	Yes	98
E	95	No	10	15	4.0	No	83
F	95	No	10	15	6.0	Yes	83
G*	85-90	Yes	25	8.5	4.0	No	76
H*	85-90	Yes	25	8.5	6.0	Yes	76
I*	85-90	Yes	20	8.5	4.0	No	68
J*	85-90	Yes	20	8.5	6.0	Yes	68
K*	95	Yes	10	8.5	4.0	No	53
L*	95	Yes	10	8.5	6.0	Yes	53
M	85-90	Yes	20	2	4.0	No	39
N	85-90	Yes	20	2	6.0	Yes	39
O	95	Yes	10	2	4.0	No	24
P	95	Yes	10	2	6.0	Yes	24
Q	97.5	Yes	5	2	4.0	No	17
R	97.5	Yes	5	2	6.0	Yes	17

*These treatment levels represent a split flow treatment scheme.

**UOD = $(1.5 \times BOD_5) + (4.5 \times NH_3\text{-N})$

levels does not dictate a particular treatment process or combination of processes since the treatment levels can be achieved with a variety of commonly used unit processes as indicated in Table 4.

The following sections of this paper incorporate the industrial point source allocation requirements and discrete treatment levels for municipal and semi-public dischargers into an overall waste load allocation procedure.

WASTE LOAD ALLOCATION METHODOLOGY

The development of a methodology for establishing waste load allocations involves consideration of several aspects of river basin management. A flow diagram of a general waste load allocation process is provided in Figure 1. Considerations related to this process which must be addressed include:

- Discharger flow regimes to be used.
- Allocation of reserve assimilative capacity for future discharges.
- Treatment levels to be examined for municipal and semi-public dischargers located on water quality limited segments.
- Industrial effluent limitations
- Methods for allocating assimilative capacity among multiple dischargers.
- Techniques for estimating model input data.

Based upon an evaluation of the results of a recent study, the general waste load allocation process has been adapted into a specific methodology designed to meet water quality standards through implementation of the least stringent treatment levels by industrial, municipal, and semi-public dischargers. Two major objectives to be met in achieving this goal are to:

- Provide an equitable allocation of assimilative capacity among existing dischargers.

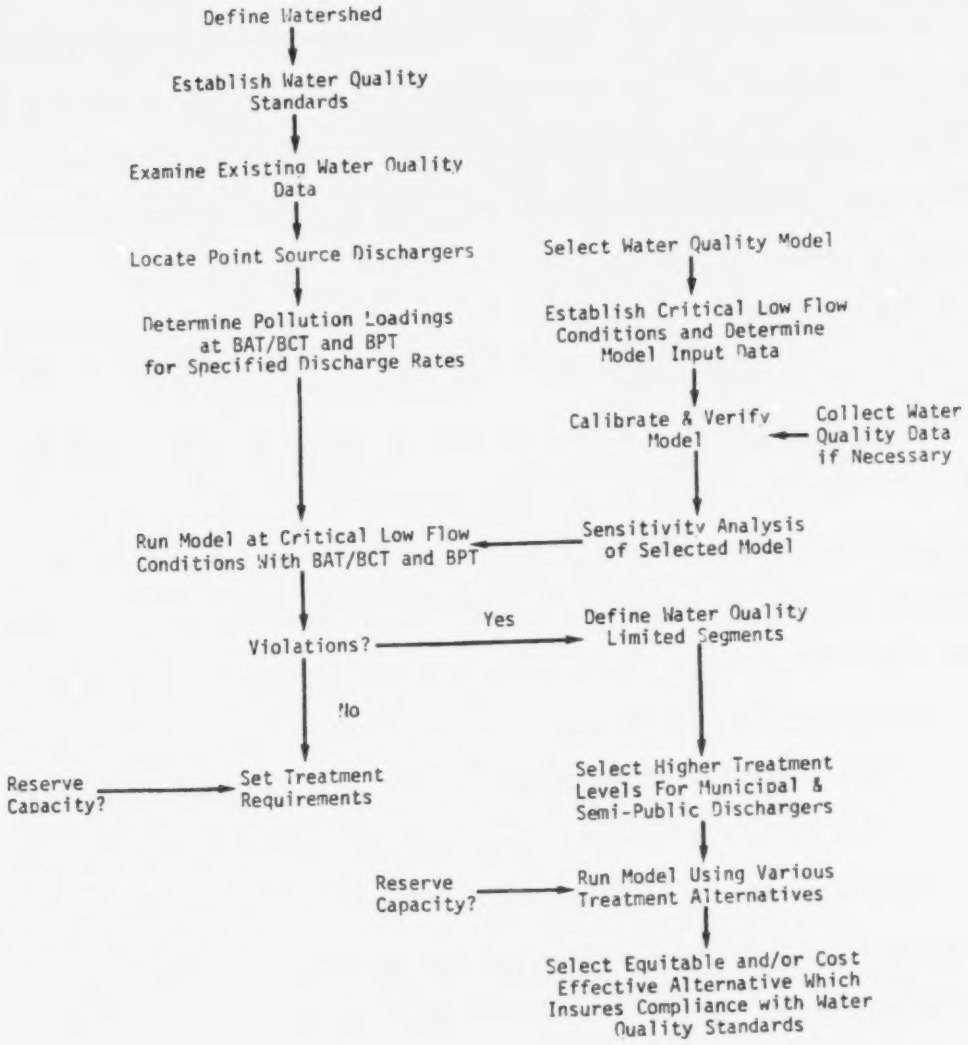
TABLE 4 ALTERNATIVE WASTEWATER TREATMENT SCHEMES

BOD_5 Removal	NH_3-N Removal	Alternative Treatment Schemes	Effluent Concentrations (mg/l)		
			BOD_5	NH_3-N	UOD
85-90%	No	1. AS + CHL	30	15	113
85-90%	Yes	1. AS + BN + CHL	20	2	39
		2. EA + CHL	20	2	39
95%	No	1. AS + MMF + CHL	10	15	83
95%	No	1. AS + LC1 + MMF + CHL	5	15	75
		2. LC1 + MMF + AC + CHL	5	15	75
95%	Yes	1. AS + BN + MMF + CHL	5	2	17
		2. AS + LC2 + AST + MMF + CHL	5	2	17
		3. LC2 + AST + MMF + AC + CHL	5	2	17
95%	Yes	1. EA + MMF + CHL	10	2	24
		2. AS + BND + AL + CHL	10	2	24
99%	Yes	1. AS + BND + AL + MMF + AC + CHL	2	2	12
		2. AS + LC2 + AST + MMF + AC + CHL	2	2	12
		3. LC2 + AST + MMF + AC + CHL	2	2	12

Abbreviations:

AC - Activated Carbon
AL - Alum in Act. Sludge
AS - Activated Sludge
AST - Ammonia Stripping
BN - Biological Nitrification
BND - Biological Nitrification and Denitrification
CHL - Chlorination
EA - Extended Aeration
LC1 - Lime Coagulation - 1 stage
LC2 - Lime Coagulation - 2 stage
MMF - Mixed Media Filtration

FIGURE 1: WASTE LOAD ALLOCATION PROCESS


Define Watershed
Establish Water Quality Standards
Examine Existing Water Quality Data
Locate Point Source Dischargers
Determine Pollution Loadings at BAT/BCT and BPT for Specified Discharge Rates
Select Water Quality Model
Establish Critical Low Flow Conditions and Determine Model Input Data
Calibrate & Verify Model
Collect Water Quality Data if Necessary
Sensitivity Analysis of Selected Model
Run Model at Critical Low Flow Conditions With BAT/BCT and BPT
Violations?
Yes
Define Water Quality Limited Segments
No
Reserve Capacity?
Set Treatment Requirements
Select Higher Treatment Levels For Municipal & Semi-Public Dischargers
Reserve Capacity?
Run Model Using Various Treatment Alternatives
Select Equitable and/or Cost Effective Alternative Which Insures Compliance with Water Quality Standards

- Provide a means for accommodating new dischargers and future flow increases from existing dischargers.

The recommended waste load allocation procedure is outlined in Figure 2. This procedure was synthesized from a recent study in which model input parameters related to reaeration and deoxygenation were determined for stream segments on several streams.

Several significant aspects of the recommended procedure warrant further discussion. Using stream sampling results obtained in a recent study, an analysis of the initial recommended values for the deoxygenation rate constants in relation to other values determined through model calibration indicated that:

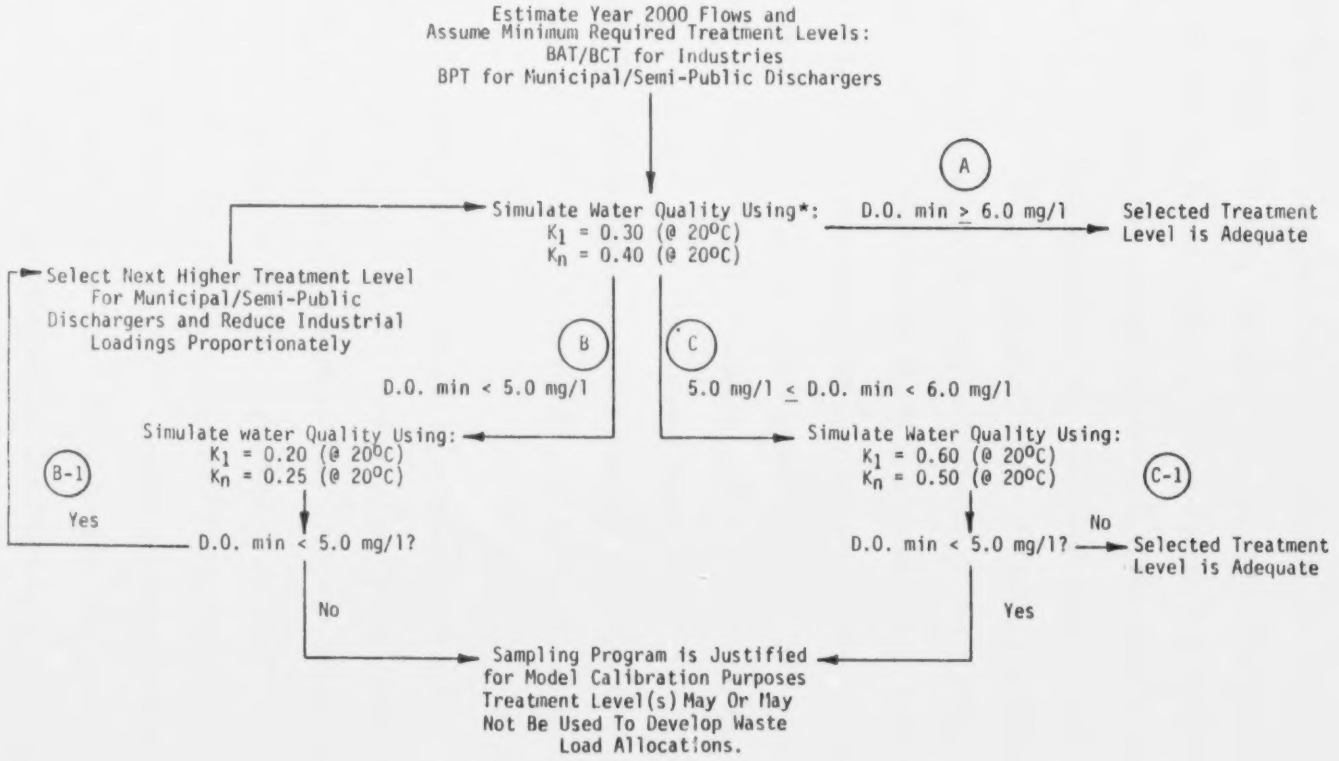
- Ninety (90) percent of the K_1 values determined for stream reaches downstream from dischargers were less than or equal to 0.30 (at 20°C).
- Seventy-five (75) percent of the Kh values determined for stream reaches downstream from dischargers were less than or equal to 0.40 (at 20°C).

Since these values appear to be conservative, they may result in overly restrictive effluent limitations. Therefore, consideration of model output using lower values may be warranted as indicated in the recommended waste load allocation procedure.

The minimum dissolved oxygen criteria associated with path A (i.e., 6.0 mg/l) was selected on the basis of the results of a sensitivity analysis of a calibrated model and past experience in water quality modeling. It appears that the achievement of this minimum dissolved oxygen level, using the initial values of the deoxygenation rate constants, will insure that water quality standards are maintained at the discharge levels in question.

The primary incentive for following path B is to justify the necessity of requiring higher treatment levels. The deoxygenation rate constants associated with this path were selected on the basis

FIGURE 2: RECOMMENDED PROCEDURE FOR DEVELOPING WASTE LOAD ALLOCATIONS ON UNCALIBRATED STREAM SEGMENTS


Estimate Year 2000 Flows and
Assume Minimum Required Treatment Levels:
BAT/BCT for Industries
BPT for Municipal/Semi-Public Dischargers
Simulate Water Quality Using*:
$K_1 = 0.30$ (@ 20°C)
$K_n = 0.40$ (@ 20°C)
A
D.O. min ≥ 6.0 mg/l
Selected Treatment Level is Adequate
B
D.O. min < 5.0 mg/l
C
5.0 mg/l ≤ D.O. min < 6.0 mg/l
Select Next Higher Treatment Level For Municipal/Semi-Public Dischargers and Reduce Industrial Loadings Proportionately
Simulate water Quality Using:
$K_1 = 0.20$ (@ 20°C)
$K_n = 0.25$ (@ 20°C)
B-1
Yes
D.O. min < 5.0 mg/l?
No
Simulate Water Quality Using:
$K_1 = 0.60$ (@ 20°C)
$K_n = 0.50$ (@ 20°C)
C-1
D.O. min < 5.0 mg/l?
No
Selected Treatment Level is Adequate
Yes
Sampling Program is Justified for Model Calibration Purposes Treatment Level(s) May Or May Not Be Used To Develop Waste Load Allocations.

*Use annual Q_{10-7} flow for winter conditions and seasonal Q_{10-7} flow for summer conditions.

of an evaluation of calibration results and other reported values. These values are representative of the lowest values which are likely to occur on the streams which were sampled. Although lower values were found to occur on some streams, primarily during summer conditions, these values appear to represent a net change in carbonaceous oxygen demand resulting from oxidation and algal contributions.

The primary incentive for following path C is to insure that the selected treatment levels are adequate to maintain water quality standards. The deoxygenation rate constants associated with this path were selected on the basis of an evaluation of calibration results and other reported values. Although higher values were determined for streams during winter-ice cover conditions, these values may not be reliable since simulated travel times may have contained significant errors under these conditions. It appears that the deoxygenation rate constant values associated with this path are the highest values which are likely to occur on the streams which were studied.

It is recommended that waste load allocations be based on estimated year 2000 flows for all dischargers to provide reserve assimilative capacity for projected future growth in the planning period. These estimated flows should include all municipalities which may construct collection and disposal systems during the twenty year planning period. This will insure reserve assimilative capacity in the receiving streams. Although waste load allocations can only be predicted and implemented for the next permit period due to possible future technology changes, water quality standards changes, etc., they are acceptable from a facilities planning standpoint. In addition to providing reserve capacity for projected growth, the use of year 2000 flows rather than 1983 flows is recommended for several additional reasons. First, since any plant constructed or upgraded in the near future will be required to treat year 2000 waste flows (i.e., the twenty year planning period), plant design should be based upon year 2000 population projections. Secondly, our experience in waste load allocation studies has indicated that treatment requirements based upon 1983 flows seldom differ from treatment requirements based upon year 2000 flows when using a treatment level approach. In those instances when treatment levels may vary, it has usually been found that it is more cost-effective to design the plant for the more stringent treatment level rather than upgrade the facility at some point during the twenty year planning period. The recommended municipal/semi-public discharger treatment levels were previously presented in Table 3.

Initially, a computer simulation of water quality should be made using BAT/BCT for industrial dischargers and BPT (i.e., Treatment Level A) for municipal and semi-public dischargers at projected year 2000 flows as inidcated in Figure 2. If no water quality violations occur (i.e., path A) then treatment requirements should be set according to BAT/BCT and BPT guidelines.

If a violation of water quality standards occurs (i.e., path B) in the case of a single discharger (municipal or semi-public), successively higher treatment levels (i.e., Treatment Levels B through R from Table 3) should be modeled using the deoxygenation rate constants indicated in Figure 2 until no violations occur (path C or A). Waste load allocations are then established based on the appropriate treatment level required to maintain water quality standards. In such cases, land application and/or controlled discharge of secondary effluent (i.e., Treatment Level A) should be evaluated on a cost-effective basis as an alternative to providing more stringent treatment requirements.

If implementation of the controlled discharge option appears feasible, it will be necessary to determine the streamflow conditions during which discharges will be permitted. Assuming that a detention lagoon with six months of storage capacity is provided, the minimum rate of discharge to the stream will be twice the average daily wastewater flow. Since it may be possible to discharge over a shorter time period (three months for example) a correspondingly higher minimum rate of discharge would be required. In order to insure that water quality standards are maintained during the period of discharge, the Q_{10-7} flow will have to be determined for that period. These streamflow and wastewater discharge rates, along with the effluent characteristics associated with Treatment Level A should then be used to simulate water quality with the initial values of the deoxygenation rate constants (i.e., $K_1 = 0.30$ and $K_n = 0.40$ at 20^oC). If no water quality violations occur, then the controlled discharge option is a feasible alternative for maintaining water quality standards. Since this option will usually be considered only for very small dischargers, it does not appear to be necessary to repeat the entire recommended waste load allocation procedure depicted in Figure 4 when considering the controlled discharge option.

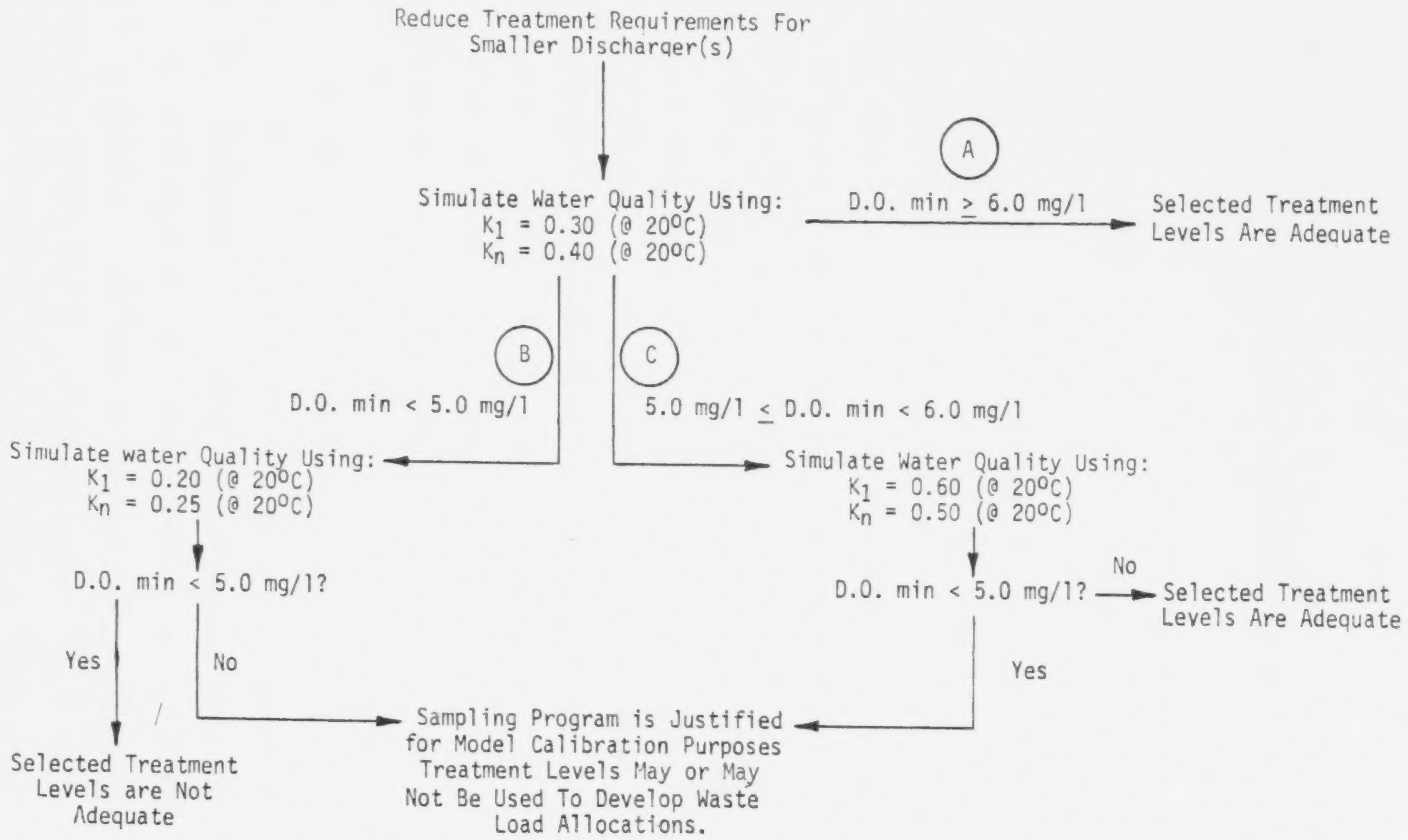
If a single industrial discharger causes water quality standards violations with BAT/BCT, then treatment must be upgraded as necessary to comply with water quality standards. It is recommended that this be accomplished by reducing the BOD_u and NH_3-N levels by the same proportion as indicated by the municipal/semi-public discharger treatment levels contained in Table 3.

In situations where the recommended procedure indicates that a water quality sampling program is justified, it should be left up to the discretion of the regulatory agency as to whether the treatment levels resulting in this destination should be used to establish waste load allocations. In such situations consideration should be given to the magnitude of the minimum dissolved oxygen levels obtained. For example, if in following path C, a minimum dissolved oxygen level of 4.90 mg/l is obtained with the second simulation (i.e., at the higher deoxygenation rate constants), the treatment levels may be considered adequate for waste load allocation purposes.

Particular attention should be given to situations in which modeling indicates that implementation of BAT/BCT and BPT is not adequate to maintain water quality standards on stream segments containing multiple dischargers. It may be possible to establish equitable waste load allocations by requiring one or more dischargers to provide more stringent treatment than other dischargers. It is recommended that this possibility be investigated only in cases where the recommended procedure leads to either path A or path C-1 with uniform treatment levels for all dischargers. In these cases, the procedure outlined in Figure 3 should be followed. This procedure which is similar to that in Figure 2, is intended to indicate those treatment level combinations which are adequate to maintain water quality standards.

As indicated in Figure 3, it is recommended that treatment levels be downgraded for the smaller dischargers rather than the larger dischargers. The objective of this approach is to require the largest dischargers to provide the most stringent treatment, since this usually results in the most equitalbe, though not

FIGURE 3: MULTIPLE DISCHARGER PROCEDURE


Reduce Treatment Requirements For Smaller Discharger(s)
Simulate Water Quality Using:
$K_1 = 0.30$ (@ 20°C)
$K_n = 0.40$ (@ 20°C)
A
D.O. min $\geq$ 6.0 mg/l
Selected Treatment Levels Are Adequate
B
D.O. min < 5.0 mg/l
C
5.0 mg/l $\leq$ D.O. min < 6.0 mg/l
Simulate water Quality Using:
$K_1 = 0.20$ (@ 20°C)
$K_n = 0.25$ (@ 20°C)
Simulate Water Quality Using:
$K_1 = 0.60$ (@ 20°C)
$K_n = 0.50$ (@ 20°C)
D.O. min < 5.0 mg/l?
Yes
No
Selected Treatment Levels are Not Adequate
D.O. min < 5.0 mg/l?
No
Selected Treatment Levels Are Adequate
Yes
Sampling Program is Justified for Model Calibration Purposes Treatment Levels May or May Not Be Used To Develop Waste Load Allocations.

always the most cost-effective alternative. In cases where water quality standards will be maintained by requiring either the largest discharger, or one or more of the smaller dischargers to provide a higher level of treatment, consideration must be given to the greater operation and maintenance capabilities of the largest discharger. In cases such as this, where multiple municipal dischargers are in close proximity and it is not cost-effective and/or equitable to provide more stringent treatment for the largest discharger, consideration should be given to cost sharing between the large and small dischargers to obtain an equitable solution. Again, land application and/or controlled discharge should be evaluated on a cost-effective basis as an alternative to requiring higher treatment levels.

Implementation of the recommended procedure for multiple dischargers in many cases will result in industrial dischargers having less stringent treatment requirements than municipalities since they are generally smaller dischargers. However, in most situations the industrial discharges of deoxygenating wastes have a much less significant impact on the dissolved oxygen levels of receiving streams than the municipal discharges. Since this may sometimes represent an undesirable situation, alternative treatment level combinations may be evaluated according to the procedure outlined in Figure 3.

As in the case of single dischargers, it should be left to the discretion of the state regulatory agency as to whether treatment levels which indicate that a sampling program is justified should be used to establish waste load allocations.

With regard to new dischargers, the recommended waste load allocation procedure provides reserve assimilative capacity in two ways:

- The use of treatment levels provides reserve capacity since they are usually more stringent than waht are actually required to maintain water quality standards.
- The use of year 2000 flows for existing and anticipated dischargers provides reserve capacity for projected future growth in the planning period.

In cases of proposed new dischargers, the recommended waste load allocation procedure should be repeated to determine the treatment level required for the new discharger to maintain water quality standards. New municipal and semi-public dischargers will not usually be significant since flows are usually small. These types of discharges should be accounted for in the year 2000 flow projections. In most cases, the only significant new dischargers in a watershed will be industries, which should consider the availability of assimilative capacity in the evaluation of site selection alternatives. The waste load allocations for existing dischargers should not be changed during a planning period as a result of new dischargers to a stream segment. Waste load allocations for new dischargers should be based on whatever treatment level is required to maintain water quality standards for the duration of the planning period. This is the most suitable and cost-effective method for accommodating new dischargers for waste load allocation purposes.

CONCLUSION

Implementation of the recommended approach will enable a state regulatory agency to reliably determine the least stringent treatment levels which are capable of maintaining water quality standards.

In addition, this approach enables the systematic identification of stream segments on which water quality sampling programs should be conducted. In consideration of the state-of-the-art of water quality modeling, the recommended approach will enable state regulatory agencies to make the best possible use of available data and appropriate water quality models for the purpose of developing waste load allocations.

CUM'S SIMPLIFIED HYDROGRAPH METHOD FOR DETERMINING URBAN RUNOFF

by C. Mitci*, H.N. Nguyen Van*, M. Osseyrane*

INTRODUCTION

The "Communauté urbaine de Montréal" (CUM) is a metropolitan services special district under whose jurisdiction the "Service d'assainissement des eaux" is responsible for the design, construction and operation of the water pollution control works on the island of Montreal, which consists mainly of two major interceptors feeding a single sewage treatment plant.

Since 1972 CUM started to investigate various methods of runoff determination capable of providing outflow hydrographs from main sewage collectors, which could be used in the rational design of the intercepting system and the real-time control of combined sewer overflows.

The main objective of the conducted study was to find a simple method of runoff determination which would:

- simulate the rainfall - runoff process;
- be sufficiently accurate and yet feasible for operating within the computational limitations of real-time control systems;
- be easily implemented on a minicomputer;
- require few and easily obtainable input data;
- not be significantly affected by the lumping of subbasins;
- become a reliable tool for urban drainage studies after calibration and verification.

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REVIEW OF THE CHARACTERISTICS OF EXISTING MODELS

A brief review of the best known methods and computer-based mathematical models developed in the last 15 years for computing storm runoff from urban areas (RRL, SWMM, ILLUDAS...) [1] , and used in practice in their original, modified or improved form revealed the following common features:

They are based on a "micro-approach" of the runoff phenomenon, which means that the main hydrological and hydraulic components of the rainfall-runoff process are analysed in detail, in particular the abstractions from rainfall and the hydraulic stages of overland, gutter and conduit flows. The process often includes the synchronizing, routing and combining of hydrographs from upstream points of entry to downstream points of concentration, both in the lateral and main sewers of the drainage subbasins.

Such an approach, however, requires considerable detail and subsequent effort in the preparation of input data, unless large lumping in catchment discretization is applied, in which case, the lumped hydraulic components (overland width, slope, gutter and conduits) do not truly represent the physical aspects of the catchment and thus, the sophisticated approach is somewhat altered by the rough representation of the basin.

On the other hand, the recent conceptual models (linear systems, kinematic models) [3 , 4] and the somewhat older but simpler synthetic unit hydrograph methods [2] derived mainly from natural watersheds and in which the physiographic properties of the basin are related to the characteristics of the resulting runoff hydrograph, generally do not have a well established antecedent in urban drainage practice.

CUM'S APPROACH

The above considerations and the main objective of the CUM to find a simple hydrograph method led to the investigation of the possibility of using a "macro-approach" to the study of the rainfall-runoff events.

In this approach, all the hydrological components of runoff (infiltration and depression storage) are grouped together through the use of a single parameter, while the hydraulic components (overland, gutter and conduit flow) are lumped together by using a direct correlation between excess rainfall and runoff as defined by the synthetic unit hydrograph theory [2] .*

However, some preliminary attempts to apply such an approach to the hydrological components, by employing an overall "runoff coefficient" varying with time [5 , 7] and accounting not only for the type of the surface but also for the observed time delay between the start of rainfall and the beginning of runoff [6], did not give satisfactory results when applied to a selected pilot basin. This was apparently due to the fact that the effects of storm pattern on depression storage and on infiltration are difficult to be represented by an overall coefficient.

Detailed investigation of models using the "micro-approach" reveals that the complicated and time consuming part of the calculations of the runoff process lies mainly in the hydraulic phase, while the hydrological phase can be easily modelled. Consequently, a "macro-approach" was adopted for the hydraulic phase and a "micro-approach" maintained for the hydrological one.

Abstractions from rainfall to determine the rainfall excess or overland flow supply rate are assumed to be mainly due to infiltration, depression storage and some minor losses resulting from evaporation and interception in the case of pervious surfaces and from evaporation, pools of water due to false slopes, losses due to surface cracks in the case of impervious areas.

* A similar approach is used in the "Battelle Urban Wastewater Management Model" [1] as outlined in an article by Brand-Stetter et al., published after the drafting of the CUM's first article on the same subject [7].

Depression storage supply is disregarded in the case of a significant antecedent precipitation.

Overland flow and depression storage supply from pervious surfaces is assumed to start when the antecedent mass rainfall equals the antecedent mass of infiltration [8] .

The rainfall excess obtained by subtracting the above losses from the recorded rainfall is directly transformed to a hydrograph by using the synthetic unitgraph concept developed by, among others, the USSCS [2] .

The extensive technical literature on the subject of the time of occurrence of peak discharge Tp [10] and the overall duration of the hydrograph reveals that this time depends on all the hydrometeorological characteristics of the rainstorm (areal and temporal distribution) and the physiographic properties of the drainage basin such as impervious surface, size, shape, a representative length, slope, surface drainage pattern, storage factor and a representative runoff resistance.

Since many of the existing empirical formulas [10] diverge considerably on the effects of the various influential parameters, it appears justified to use a simple triangular hydrograph, such as the one developed by USSCS [2], and illustrated in Fig. 1. In this hydrograph, the time of peak surface runoff depends on the location of the centroid of the excess rainfall hyetograph and on a characteristic factor of the basin which is expressed by βTc. In this expression, the components of the familiar time of concentration Tc (inlet and travel time) reflect mainly the characteristics of the drainage system, such as length and slope. The factor β accounts for the other physical properties of the basin.

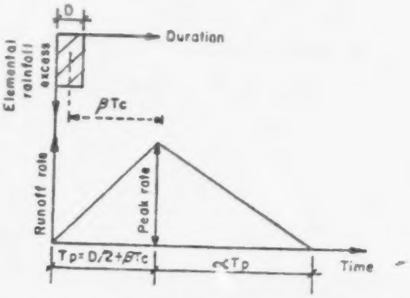

Fig. 1 Geometric elements of triangular hydrograph

In the routing of the hydrographs through the conduits the "time-offset" procedure is used as being consistent with the overall simplification sought [8].

MODELLING

The model requires the following physical data of the catchment: area, percentage of imperviousness, time of concentration and, if discretization is applied, time offset of subcatchment hydrograph to basin outlet. Values of parameters α and β of the unit triangular hydrograph should also be determined in each particular case.

The model computes runoff based on specified rainfall as follows*:

1. Determination of the beginning of overland flow and depression storage supply rate from pervious surfaces, by equalizing the antecedent mass of rainfall and the antecedent mass of infiltration.

   Infiltration is expressed by Horton's exponential function:

$$f = f_c + (f_o - f_c)\, e^{-K\, t_f} \qquad (1)$$

Where: f = infiltration capacity rate,

t_f = time from the beginning of the infiltration capacity curve;

f_c = the constant rate at which f is approached asymptotically as time increases, or final sustained rate;

f_o = infiltration capacity rate when $t_f = 0$, or initial sustained rate;

---

* Linear interpolation is used when rainfall interval exceeds one minute.

K = decay rate of infiltration related to soil characteristics (1/time unit);

e = base of the system of natural logarithms.

(Values of f_c and K vary with the antecedent moisture conditions).

2. Calculation of the overland flow supply (rainfall excess) from pervious surfaces by subtracting from the rainfall depth losses due to infiltration, depressional storage and other minor losses:

$$ie = K_1 (i - f - s) \tag{2}$$

Where ie = rainfall excess;

i = rainfall intensity;

f = infiltration capacity rate

s = depression storage supply = $(i - f)\, e^{-\frac{P-F}{S_d}}$ (3)

P,F = accumulated volume of water precipitated and infiltrated, respectively;

S_d = total depression storage capacity related to basin characteristics (type of cover, slope)

e = base of the system of natural logarithms;

K_1 = a dimensionless coefficient < 1 which in urban catchments accounts for minor losses.

3. Calculation of overland flow supply (rainfall excess) from impervious surfaces by subtracting from the rainfall depth losses due to depressional storage and other minor losses. (in this case f = o).

$$ie = K_2 \ (i - s) \tag{4}$$

Where: $$s = i \ e^{-\frac{P}{S_d}} \tag{5}$$

K_2 = dimensionless coefficient ≤ 1 which accounts for minor losses.

4. Transformation of the rainfall excess hyetographs determined in steps 2 and 3, to runoff hydrographs at the outlet of each subbasin by applying the triangular hydrograph concept to elemental excess rainfall hyetographs of short duration (one minute).

5. Combination of the hydrographs obtained in step no 4 at the outlet of each subbasin.

6. Routing of the combined hydrographs of step no 5, by the "time-offset" method, through the main conduits of the catchment in order to obtain ultimately a computed outflow hydrograph at the outfall of the basin.

7. In case of combined sewers, addition of a specified base-flow (dry weather flow) to the calculated runoff to obtain the total outflow of the catchment.

Fig. 2 illustrates the above steps.

PROGRAM DESCRIPTION

The program is written in FORTRAN IV language and is implemented on a Digital PDP 11/34 minicomputer. It can also be executed on an IBM 370 without any modification.

It has about 300 statements and takes 14 K of memory without overlays. The time of execution is less than half a minute to simulate an event of five hours.

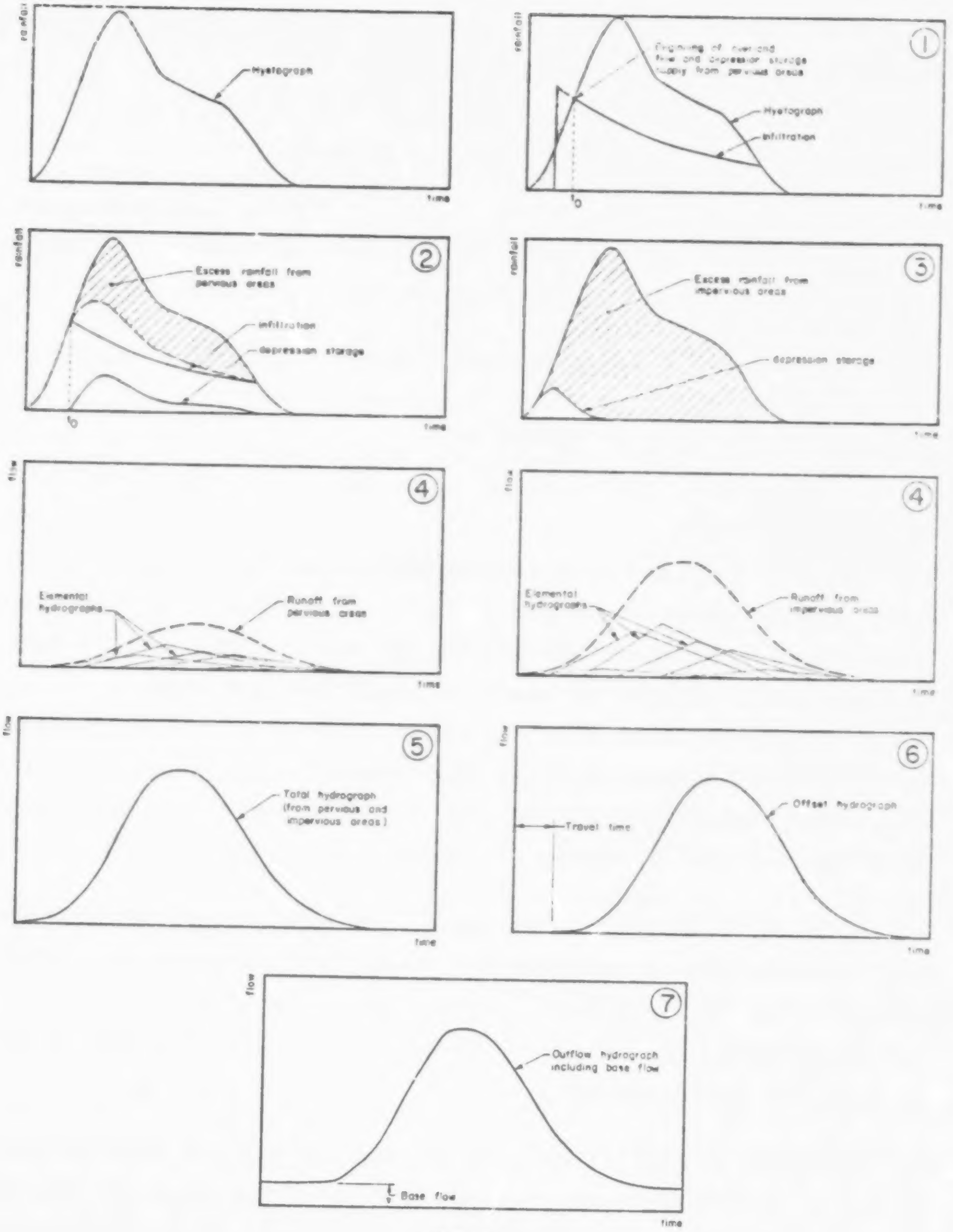
rainfall
Hyetograph
time
rainfall
①
Beginning of overland flow and depression storage supply from pervious areas
Hyetograph
Infiltration
t_0
time
rainfall
②
Excess rainfall from pervious areas
infiltration
depression storage
t_0
time
rainfall
③
Excess rainfall from impervious areas
depression storage
time
flow
④
Elemental hydrographs
Runoff from pervious areas
time
flow
④
Elemental hydrographs
Runoff from impervious areas
time
flow
⑤
Total hydrograph (from pervious and impervious areas)
time
flow
⑥
Travel time
Offset hydrograph
time
flow
⑦
Outflow hydrograph including base flow
Base flow
time

FIG. 2

ILLUSTRATIONS OF THE VARIOUS CALCULATION STEPS OF THE C.U.M. MODEL

DESCRIPTION OF THE SELECTED TEST BASINS

Three urban drainage basins of different size, location and characteristics were selected in order to verify the proposed method of runoff determination. (Fig. 3, 4 and 5).

No	Basin	Location	Size (acres)	Approximate population	% of imperviousness	Time of concentration (min)
1	Ball-De l'Epée	Montreal	70	6,000	75	21
2	Curotte-Papineau	Montreal	2214	103,000	47.5	50
3	Malvern	Burlington (Ontario)	58	1,000	34	12

The general surface topography of basin nos 1 and 2 is characterized by a gentle slope. In basin no 1, land use is predominantly residential and consists mainly of row duplexes with flat roofs, narrow front strips of grassed surfaces and wider grassed backyards draining into sewer inlets located in paved alleys. Institutional and commercial use together with open spaces and parks represent less than 15% of the total area of the catchment. All roof drains are directly connected to the sewers.

In basin no 2, residential land use represents approximately 60% and industrial use 20% of the total catchment area. 85% of the residential zone consists entirely of row duplexes, the rest being occupied by single-family units. The surface drainage pattern is similar to basin no 1.

In basin no 3 [12], the entire area is single family residential with no vacant lots or parks. The average size of the lots is rather large with the front and backyards alone representing approximately 65% of the total catchment area. Backyards drain into gulleys which in turn drain into streets or into sewer inlets.

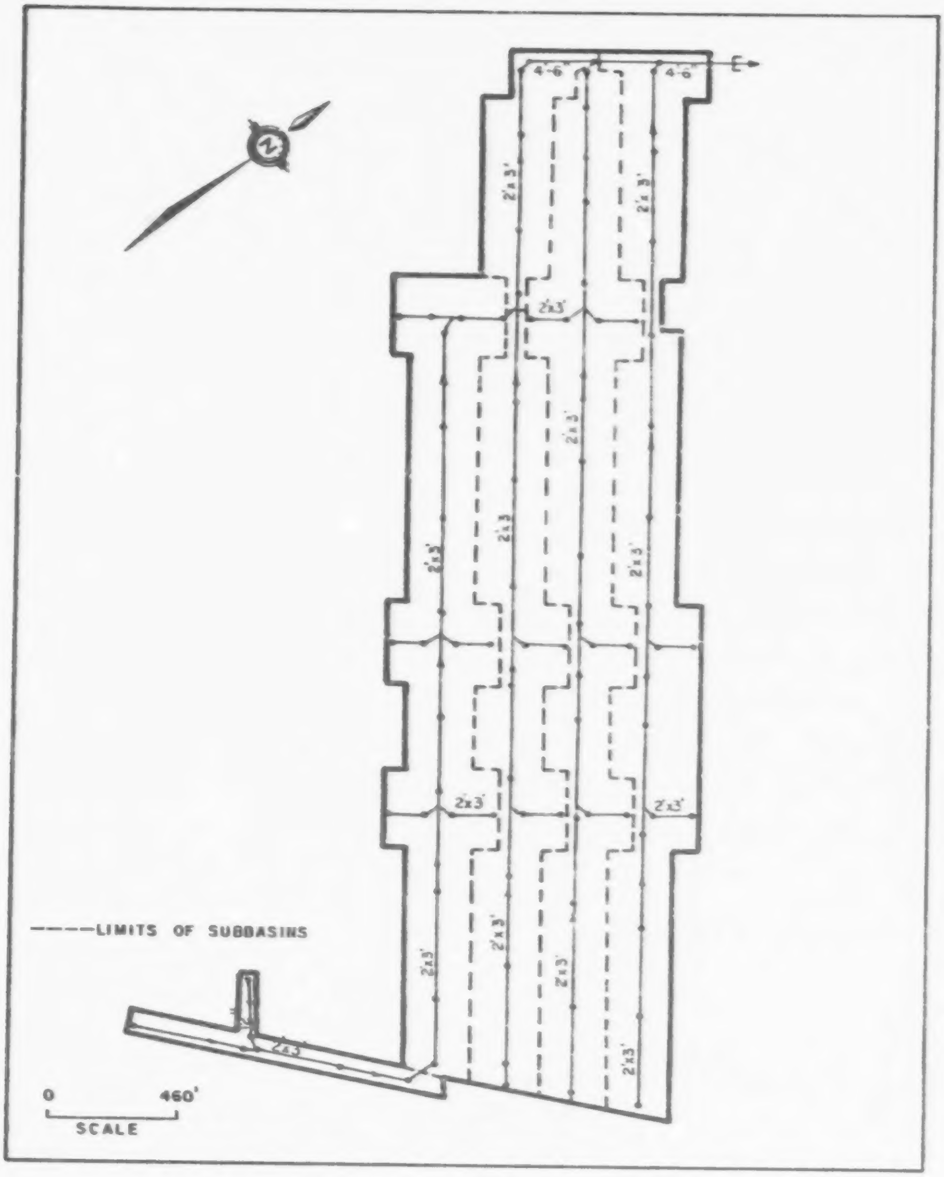
4-6'
E
4-6'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
2'x3'
N
----LIMITS OF SUBBASINS
0
460'
SCALE

FIG. 3 BASIN Nº I (BALL DE L'ÉPÉE)

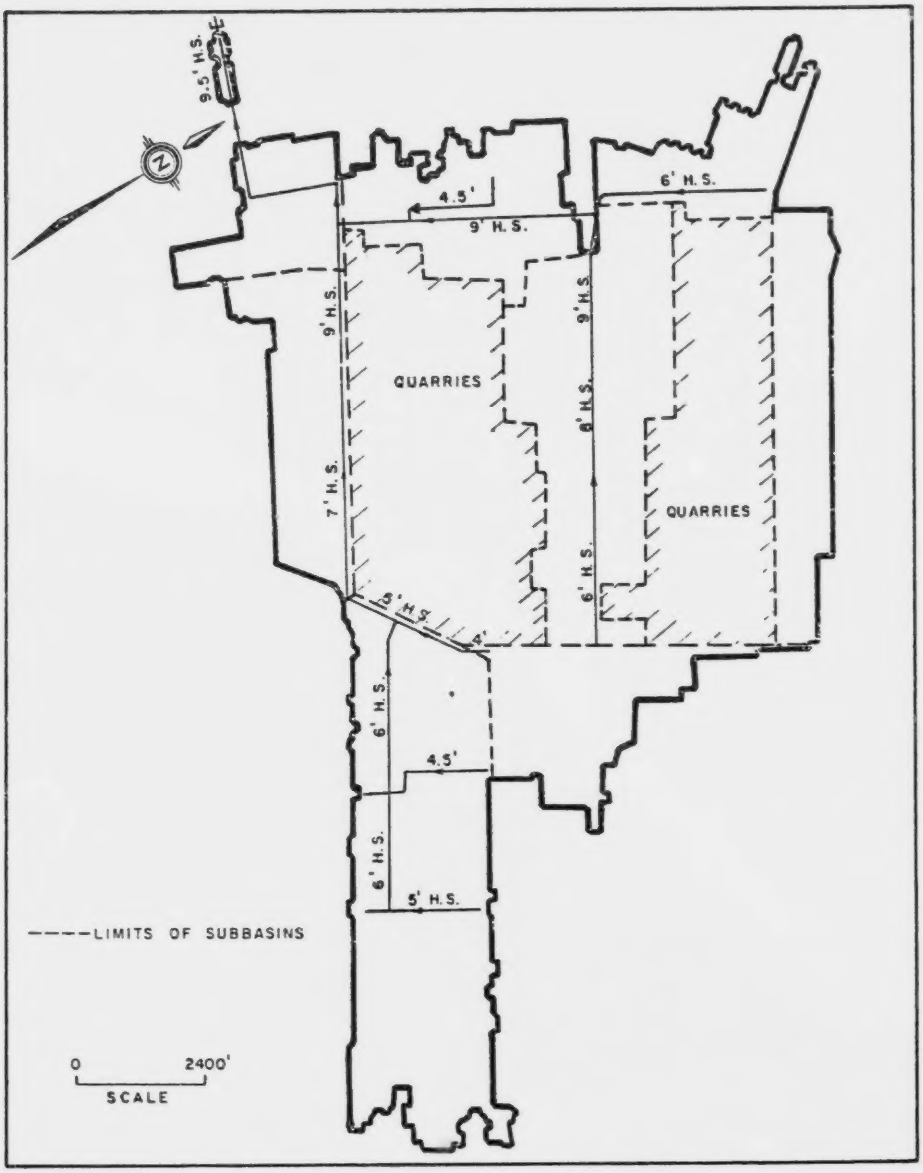
9.5' H.S.
N
4.5'
9' H.S.
6' H.S.
9' H.S.
9' H.S.
QUARRIES
8' H.S.
7' H.S.
QUARRIES
6' H.S.
5' H.S.
4'
6' H.S.
4.5'
6' H.S.
5' H.S.
----LIMITS OF SUBBASINS
0
2400'
SCALE

FIG. 4 BASIN Nº 2 (CUROTTE PAPINEAU)

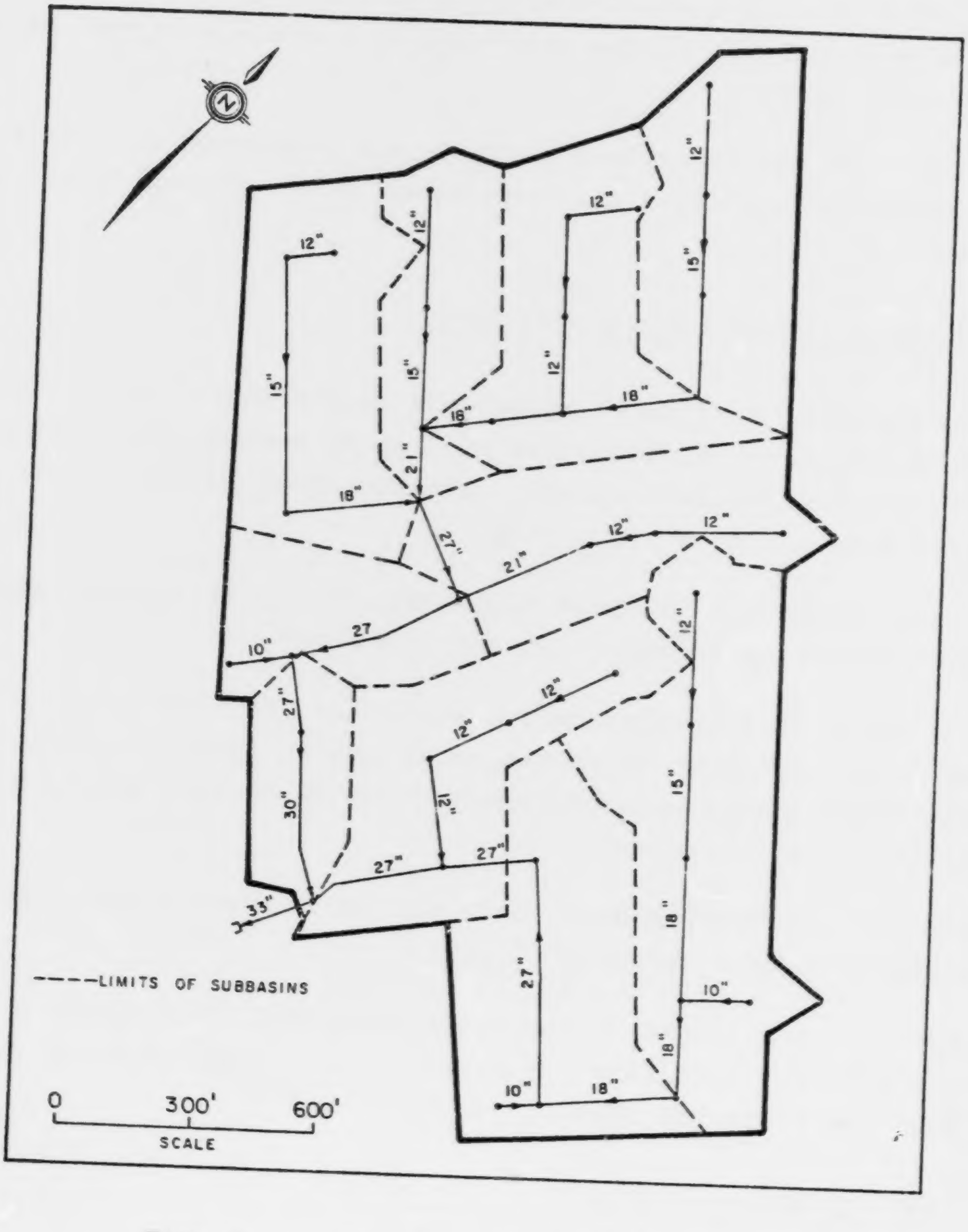
N
12"
15"
12"
15"
16"
21"
18"
12"
12"
18"
12"
15"
27"
21"
12"
12"
10"
27
27"
30"
12"
12"
12"
27"
27"
33"
15"
12"
18"
27"
10"
18"
18"
10"
LIMITS OF SUBBASINS
0
300'
600'
SCALE

FIG. 5 BASIN Nº 3 (MALVERN)

The general surface topography is characterized by a gentle slope. Directly connected impervious areas represent 31% of the total.

Estimated time of concentration at the outlet of each of the three basins was based on full pipe travel time. The difference found between this time and travel time calculated on the basis of a flow proportional to the contributing sub-areas of the catchment is quite small.

Inlet time for impervious surfaces was evaluated according to the location of the first upstream sewer inlet and some field observations.

RECORDED RAINFALL AND RUNOFF DATA

For basin no 1, precipitation data were provided by the CUM's raingauge network consisting of tipping bucket type raingauges connected directly to a mini computer (Digital PDP 11/34) which also does the data processing.

Runoff data were obtained from stage-discharge curves established from direct measurements at the basin outlet.

For basin no 2, precipitation data were provided by the City of Montreal's own tipping bucket raingauges. Flow was measured at the basin outlet by a permanently installed weir previously calibrated in a laboratory.

Rainfall and runoff data for basin no 3 were extracted from various studies on this catchment [12, 13]

For basins no 1 and 2, input hyetograph of five-minute intervals was used for modelling. For basin no 3, this interval varies from one to ten minutes.

DISCRETIZATION OF SELECTED TEST BASINS

It is common practice in urban runoff modelling to discretize the catchment, i.e., to divide the area and the related drainage system into discrete elements.

Although no general rules for such a procedure exist, the most common assumption is the finer the discretization, the more accurate the representation of runoff phenomenon.

One of the main objectives of the CUM's method was, however, to obtain acceptable results by large lumping of the subbasins and ultimately by considering the entire basin as a single catchment.

Comparison between outflow hydrographs calculated by dividing the three test basins into 6, 8 and 10 subbasins respectively and by considering the entire basin as a single catchment, showed no significant improvement in the results obtained by discretization (Fig. 6). Therefore, a single basin approach is adopted for the CUM model.

MODEL CALIBRATION

Through calibration, which normally constitutes a prerequisite to the verification of the model, an adjustment of input parameters is made until a good fit is obtained between measured and computed hydrographs [14] .

It must be noted, however, that the adjusted values of some input parameters should not normally deviate too much from the corresponding extreme values reported in the literature, otherwise one may be left with an input that is partly man-made.

The intensity of the recorded precipitations in all three test basins was not sufficiently high to produce significant runoff from the pervious areas. Thus, the model was calibrated for the impervious areas directly connected to the sewer system.

Basin no 1 was calibrated with storm nos 2, 5, 7 and 8 (Table 1), basin no 2 with storm nos 1 to 10 (Table 2), and basin no 3 with storm nos 3, 7 and 8 (Table 3).

The following parameters of the model have to be calibrated:

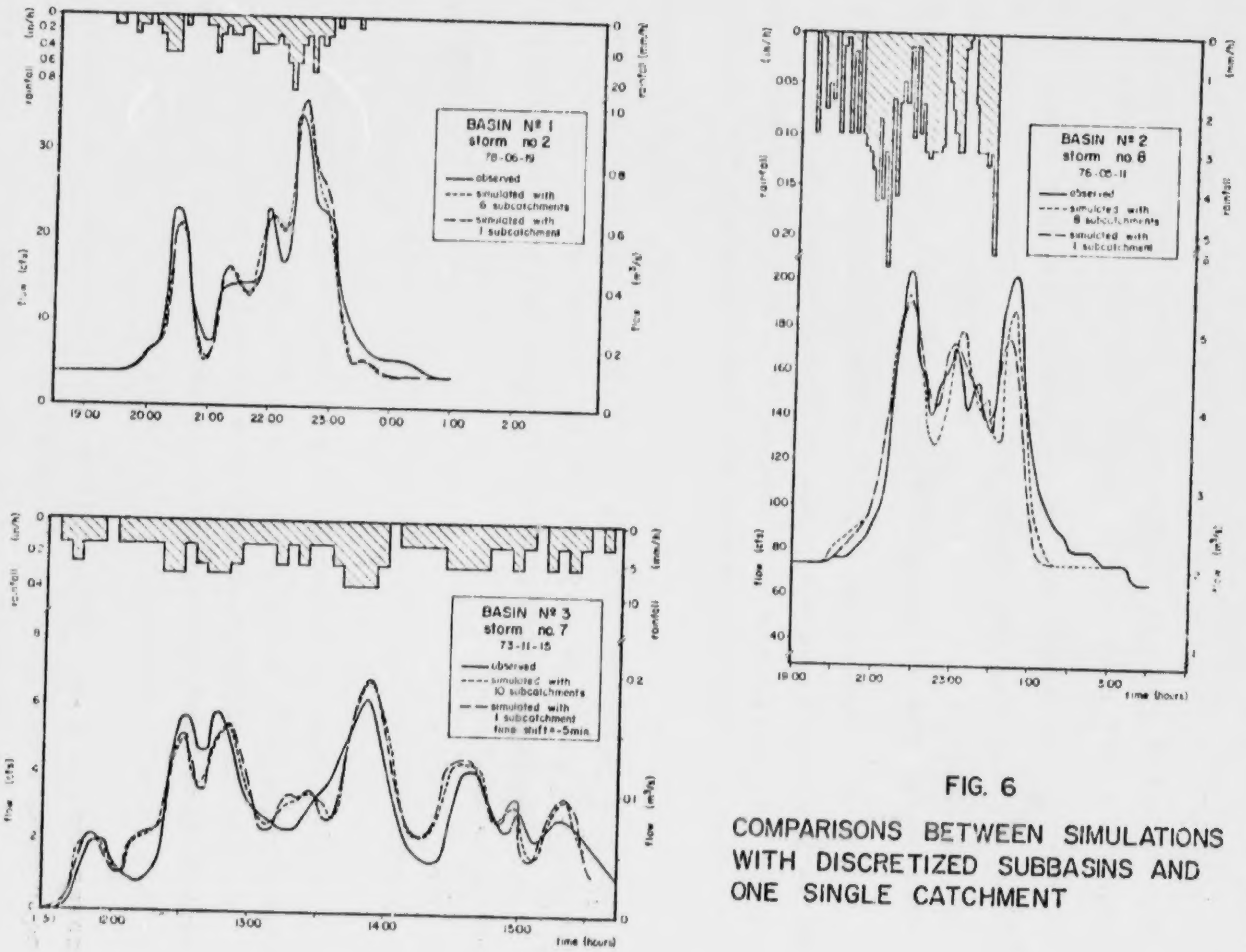

FIG. 6

COMPARISONS BETWEEN SIMULATIONS WITH DISCRETIZED SUBBASINS AND ONE SINGLE CATCHMENT

Percentage of imperviousness

This physical characteristic of the basin is particularly important since it affects directly the runoff volume. It can be obtained by direct site measurements (Basin nos 1 and 3) or estimated from typical land use (Basin no 2). In this latter case, it should be verified by comparing the total excess rainfall with the measured runoff volume.

Since the values of this parameter used for all three basins gave good results, no further ajustments have been necessary.

Depression storage capacity (Sd)

The effects of this parameter on computed peak flow and runoff volume depend on the storm pattern and the total rainfall, especially in the case of advanced storm pattern and low precipitation event.

The value used for basins no 1 and 2 is 0.0625 in. as suggested in the literature [8] and 0.020 in. for basin no 3, as found in previous studies on this catchment [12, 13].

Coefficient of minor losses (K_1, K_2)

The value of this coefficient should be close to unity. A value of 0.98 was used for all test basins.

Geometric elements of the triangular hydrograph (α and β)

Various tests showed that values of α and β derived from observed single-peak hydrographs, if used in the elemental triangular hydrograph, result in a poor fit between the computed and observed hydrographs.

Morever, no correspondence was observed between the values of the parameters used in the elemental triangular hydrograph and those of the computed hydrograph derived from it. The degree of discrepancy depends on the duration of the elemental rainfall excess hyetograph considered.

Therefore, α and β should be calibrated.

Good results were obtained with $\alpha = 1.5$ for the three test basins and with values of $\beta = 0.57$, 0.54 and 0.45 respectively for basins no 1, 2 and 3. These values are fairly close to the average ones recommended by USSCS ($\alpha = 1.67$ and $\beta = 0.6$) for ungaged catchments.

RESULTS OF SIMULATION

Simulations of 9, 10 and 23 rainfall-runoff events respectively for the three basins were made.

A summary of these events is shown in Tables 1, 2 and 3, where the mean and standard deviation of recorded/simulated peak flows and flow volumes indicate a good fit between observed and calculated hydrographs.

Some typical results are shown in Fig. 7, 8 and 9.

Using the results of various studies on basin no 3 [12, 13] , a comparison of simulation with SWMM, ILLUDAS and CUM models is presented in Table 4. It should be noted that with SWMM and ILLUDAS, discretization of basin no 3 in 10 and 40 subcatchments respectively was applied, while with the CUM model only one lumped single catchment was considered. The results show that the three models compare fairly well.

SUMMARY AND CONCLUSIONS

Recorded precipitation and runoff data in three test basins were used to calibrate and verify the proposed CUM model for urban runoff determination.

Because of the relatively low intensities of the precipitations recorded, runoff from pervious areas could not be verified.

Simulations made with a lumped single catchment for each basin showed that the CUM model, due to its nature, is not very sensitive to discretization. This suggests that it can be used with some confidence with a minimum amount of input data.

The results obtained indicate that, within the scope and purpose of this utilization, this simple, quick and inexpensive computer model could become a fairly accurate tool for predicting the quantity of runoff from urban catchments and for enabling the initiation of the real-time control of intercepting systems.

TABLE 1 SUMMARY OF SIMULATIONS ON BASIN NO. 1 (BALL-DE L'EPEE)

Event No	Date	Total rainfall (in)	Peak Flow (cfs)			Volume (ft^3)			Time to peak (min.)		
			0	S	0/S	0	S	0/S	0	S	0-S
1	78-05-31	0.48	65.33	71.26	0.9168	76739	79066	0.9706	16	23	-7
2	78-06-19	0.94	34.08	35.77	0.9528	171375	163645	1.0472	192	194	-2
3	78-06-20	1.37	28.82	27.02	1.0666	296625	253210	1.1715	305	301	4
4	78-06-27	0.46	44.21	52.12	0.8482	68739	74645	0.9209	56	64	-8
5	78-07-27	0.42	37.08	38.10	0.9732	65332	67091	0.9738	62	66	-4
6	78-08-08	0.29	20.62	18.54	1.1122	40541	41671	0.9729	29	30	-1
7	78-09-21	1.18	15.89	16.07	0.9888	36397	32645	1.1149	23	28	-5
8	78-09-28	0.34	26.06	23.63	1.1028	51675	52400	0.9862	130	131	-1
9	78-10-14	0.89	13.31	13.60	0.9787	176363	152718	1.1548	420	426	-6

0 = Observed values $\bar{x}$ = 0.9933 $\bar{x}$ = 1.0348 $\bar{x}$ = -3.33

S = Simulated values s = 0.0871 s = 0.0913 s = 3.74

Table 2 SUMMARY OF SIMULATION ON BASIN NO. 2 (CUROTTE-PAPINEAU)

Event No	Date	Total rainfall (in)	Peak flow (cfs)			Volume (ft^3)			Time to peak (min.)		
			0	S	0/S	0	S	0/S	0	S	0/S
1	76-07-24	0.35	353.1	364.3	0.9692	1 646 183	1 752 979	0.9390	95	86	9
2	75-07-27	0.63	469.6	404.9	1.1600	3 317 960	3 203 335	1.0357	130	116	14
3	75-09-02	0.95	541.7	761.0	0.7118	3 914 314	3 847 365	1.0174	230	242	-12
4	75-10-13	0.57	402.0	376.6	1.0674	2 456 859	2 870 634	0.8558	140	152	-12
5	76-07-16	0.83	646.2	679.8	0.9505	3 805 813	3 790 725	1.0039	80	75	5
6	76-07-27	0.81	776.9	823.8	0.9430	5 069 316	4 354 208	1.1642	50	52	-2
7	76-09-23	0.13	132.0	127.9	1.0320	1 197 352	1 258 724	0.9512	135	127	8
8	76-05-11	0.38	203.4	190.4	1.0682	3 041 563	3 017 141	1.0080	150	141	9
9	76-06-11	0.44	399.0	402.3	0.9917	2 832 870	2 632 679	1.0760	45	44	1
10	76-09-18	0.47	512.0	440.5	1.1623	2 231 221	2 134 785	1.0451	40	53	-13
11	75-07-25	0.27	487	452	1.0774	1 948 227	1 771 269	1.0999	55	46	9
12	75-07-28	0.30	472	441	1.0729	1 869 861	1 801 165	1.0381	55	47	8
13	75-08-21	0.10	104	122	0.8524	640 450	697 670	0.9179	135	105	30
14	75-09-02	0.21	215	204	1.0539	1 207 197	1 126 555	1.0715	100	80	20
15	75-09-06	0.31	236	259	0.9111	1 828 083	1 902 732	1.0408	55	64	9
16	75-10-15	0.17	153	135	1.1333	1 360 432	1 362 865	0.9982	85	73	12
17	75-11-21	0.20	229	214	1.0700	1 346 539	1 160 982	1.1598	90	101	-11
18	76-05-31	0.24	130	154	0.8480	2 390 457	2 449 839	0.9757	105	97	8
19	76-06-25	0.90	519	500	1.0380	5 389 108	5 103 147	1.0560	55	58	-3
20	76-07-07	0.40	485	584	0.830	1 762 713	1 777 832	1.0085	50	43	7
21	76-09-20	0.60	258	255	1.0117	3 815 687	3 908 103	0.9763	100	147	13
22	76-09-22	0.21	243	267	0.9101	1 794 179	1 757 534	1.0208	62	53	9
23	76-09-26	0.64	194	187	1.0374	4 180 745	4 339 330	1.0367	300	315	-15

0 = Observed values

S = Simulated values

Peak flow: $\overline{x}$ = 0.9957, s = 0.1130

Volume: $\overline{x}$ = 1.0215, s = 0.0705

Time to peak: $\overline{x}$ = 11.4, s = 37.15

TABLE 3 SUMMARY OF SIMULATIONS ON BASIN NO. 3 (MALVERN)

Event No	Date	Total rainfall (in)	Peak Flow (cfs)			Volume (ft^3)			Time to peak (min.)		
			0	S	0/S	0	S	0/S	0	S	0-S
1	73-09-22	0.71	32.4	29.7	1.0909	54600	44792	1.2189	42	44	-2
2	73-09-23	0.36	25.2	30.0	1.2600	25400	23474	1.0820	117	120	-3
3	73-10-13	0.31	8.4	8.4	1.0000	19900	19822	1.0039	112	111	1
6	73-11-14	0.60	10.5	9.4	1.1170	44200	39912	1.1074	344	356	-12
7	73-11-15	0.71	6.5	6.8	0.9558	46100	47494	0.9706	142	143	-1
8	73-11-28	0.47	9.5	9.9	0.9595	32800	34249	0.9576	27	45	-18
9	#4 1974	0.63	31.8	34.9	0.9111	44717	41295	1.0827	34	32	2
10	#7 1974	1.20	27.2	24.0	1.1333	15925	15893	1.0020	13	11	2
11	#12 1974	0.63	15.1	20.1	0.7572	36183	43541	0.8310	13	11	2
12	#15 1974	0.30	8.8	8.4	1.0476	20283	19309	1.0504	54	54	0

0 = Observed values

S = Simulated values

Peak Flow 0/S: $\bar{x} = 1.0232$, $s = 0.1392$

Volume 0/S: $\bar{x} = 1.0306$, $s = 0.1040$

Time to peak 0-S: $\bar{x} = -2.545$, $s = 6.5$

Table 4: COMPARISONS BETWEEN SWMM, ILLUDAS & CUM SIMULATIONS

ON BASIN NO 3 (MALVERN)

Event No	Peak Flow (cfs)				Volume (ft^3)				Time to peak (min)			
	0	0/SWMM	0/ILLUDAS	0/CUM	0	0/SWMM	0/ILLUDAS	0/CUM	0	0-SWMM	0-ILLUDAS	0-CUM
1	32.4	0.9391	0.9672	1.0909	54600	1.1030	1.1294	1.2189	42	2	0	-2
2	25.2	1.1277	1.1749	1.2600	25400	1.0410	1.0686	1.0820	117	-5	-5	-3
3	8.4	1.0181	1.0696	1.0000	19900	0.9522	0.9809	1.0039	112	2	2	1
6	10.5	1.1138	1.1258	1.1170	44200	1.0676	1.0869	1.1074	344	-10	-11	-12
7	6.5	0.9377	0.9377	0.9558	46100	0.9370	0.9525	0.9706	142	-6	-3	-1
8	9.5	0.9938	1.1778	0.9595	32900	1.000	0.9724	0.9576	27	-17	-8	-18
9	31.8	0.8231	0.8508	0.9111	44717	1.1284	1.1463	1.0828	34	6	4	2
10	27.2	1.1574	1.1628	1.1333	15925	1.1045	1.1356	1.0020	13	4	4	2
11	15.1	0.8146	0.8168	0.7572	36183	0.9068	0.9203	0.8310	13	4	4	2
12	8.8	1.1794	1.2408	1.0476	20283	1.0982	1.1297	1.0504	54	0	2	0

0 observed values

	0/SWMM	0/ILLUDAS	0/CUM	0/SWMM	0/ILLUDAS	0/CUM	0-SWMM	0-ILLUDAS	0-CUM
$\bar{x}$ =	1.0104	1.0524	1.0232	1.0331	1.0523	1.0306	-2	-1.10	-2.54
s =	0.1328	0.1493	0.1392	0.0798	0.0869	0.1040	7.34	5.40	6.51

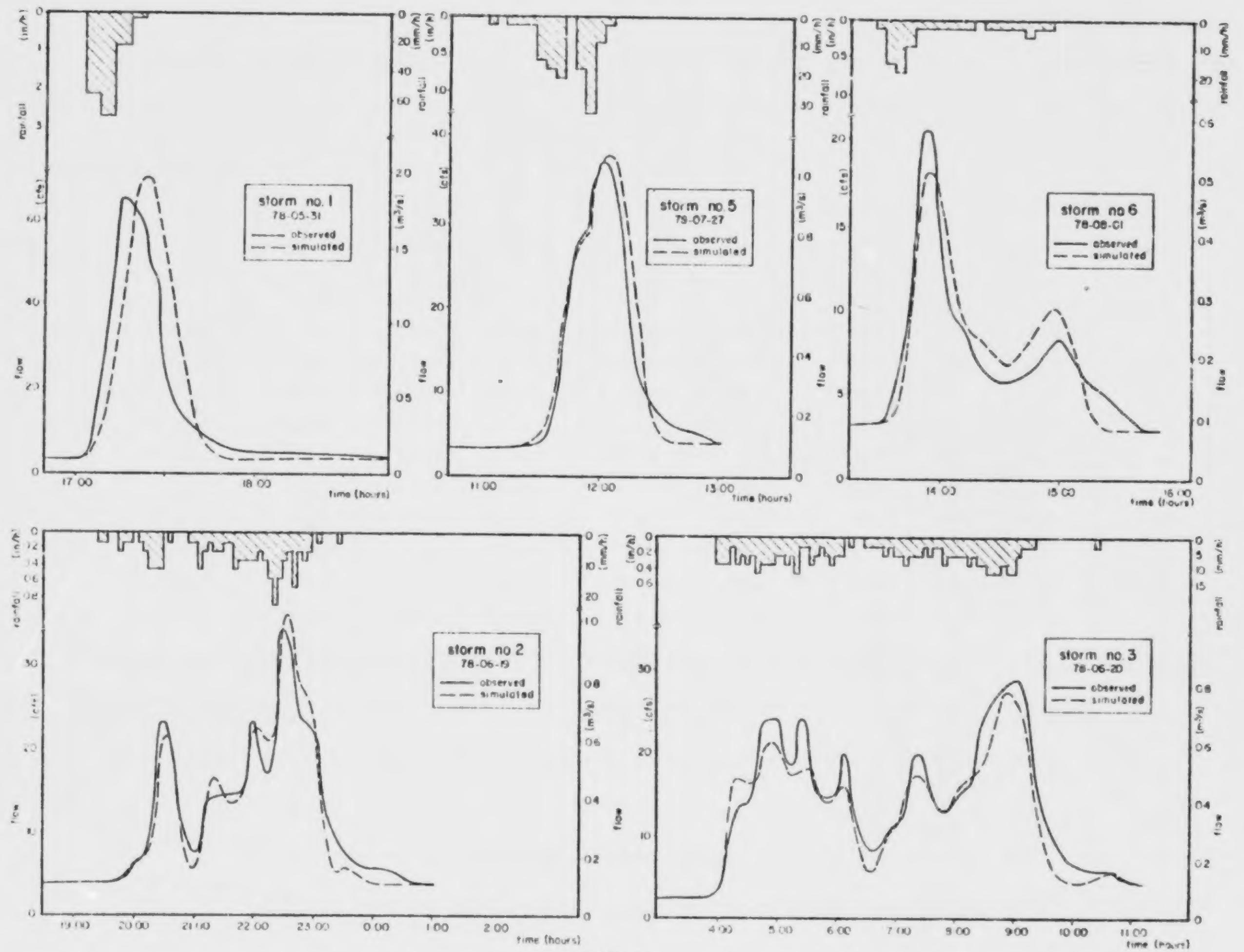

FIG. 7

BASIN Nº1 (BALL-DE L'ÉPEE)

OBSERVED AND SIMULATED HYDROGRAPHS

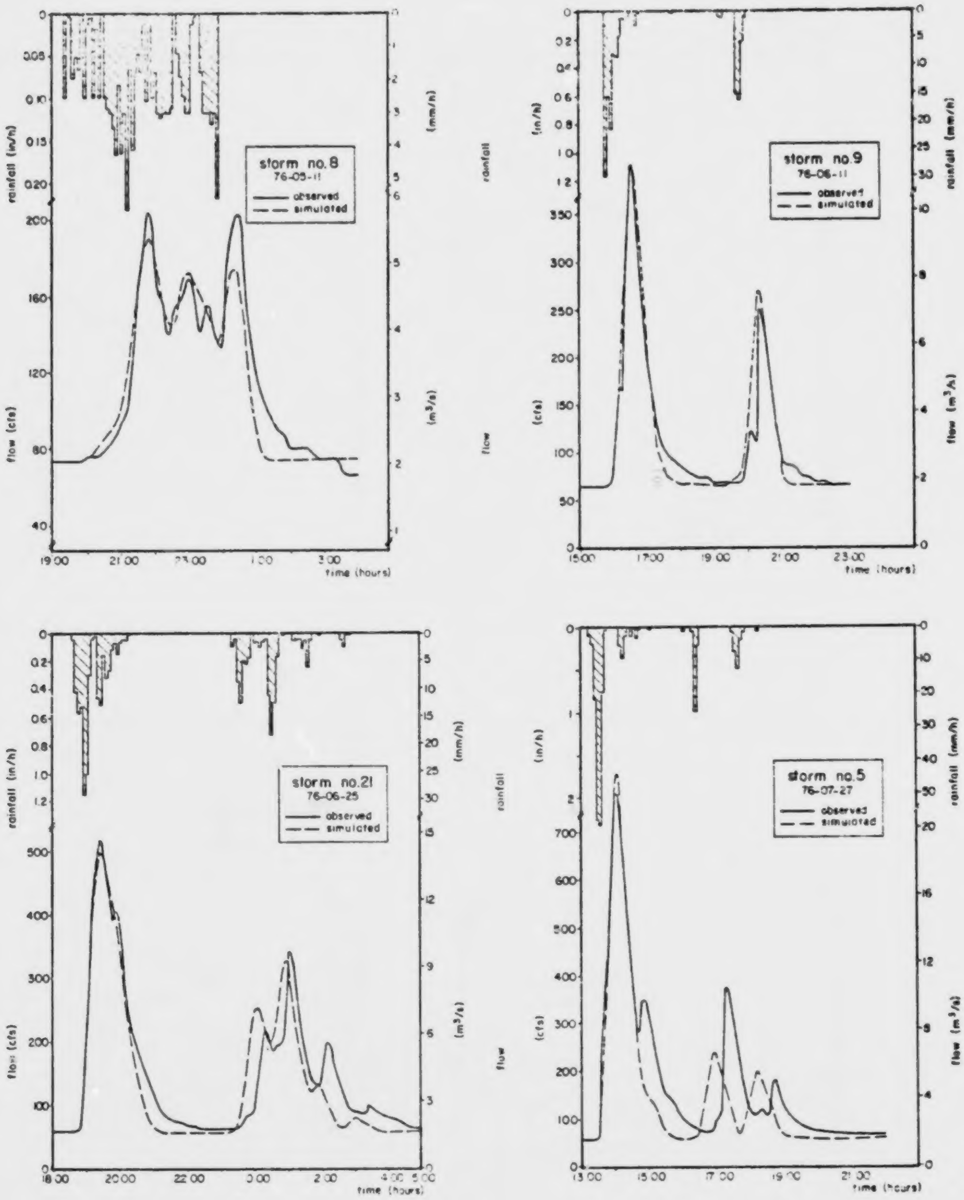

FIG. 8

BASIN N°2 (CUROTTE-PAPINEAU)

OBSERVED AND SIMULATED HYDROGRAPHS

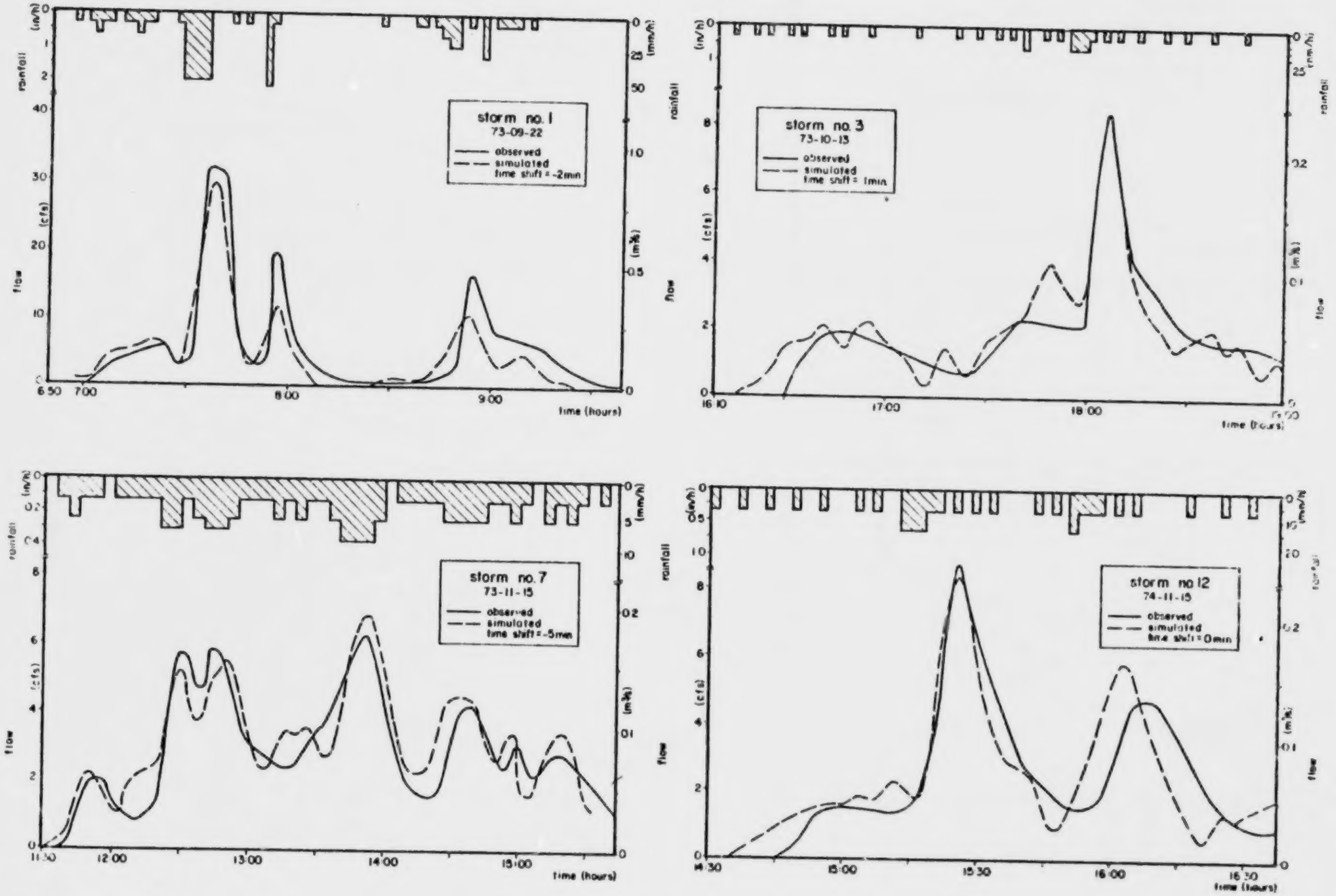

FIG. 9

BASIN No3 (MALVERN)

OBSERVED AND SIMULATED HYDROGRAPHS

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Feasibility of Storm Tracking For Automatic Control of Combined Sewer Systems

by

Van-Thanh-Van Nguyen
M.B. McPherson
Jean Rousselle

Presented at the

Stormwater Management Model (SWMM)
Users Group Meeting
Montreal, Quebec
May 25, 1979

PREFACE

by M. B. McPherson

Background

The following Technical Memorandum is Addendum 6 of a 1977 ASCE Program report on "Urban Runoff Control Planning".(1) Addendum 1, "Metropolitan Inventories," and Addendum 2, "The Design Storm Concept," were appended to the latter report. Addenda 3,(2) 4(3) and 5(4) were the first of several additional, individual additions planned for release over the period 1977-1979.

The principal intended audience of the ASCE Program's June, 1977, report was the agencies and their agents that are participating in the preparation of areawide plans for water pollution abatement management pursuant to Section 208 of the Federal Water Pollution Control Act Amendments of 1972 (P.L. 92-500). While the presentation which follows is also directed to areawide agencies and their agents, it is expected that it will be of interest and use to many others, particularly local governments.

ASCE Program

The American Society of Civil Engineers' Urban Water Resources Research Program was initiated and developed by the ASCE Urban Water Resources Research Council. The basic purpose of the Program is to promote needed research and facilitate the transfer of findings from research to users on a national scale.

Abstracts of the twenty-eight technical memoranda and reports of the Program for the 1967-1974 period are included in a readily available paper.(5) Two reports and six technical memoranda of the regular series completed since are identified in a companion publication.(6) Also included in the latter is a listing of all but one of the twelve national reports in the special technical memorandum series for the International Hydrological Programme; and the last national report(7) and an international summary(8) have been released since.

A Steering Committee designated by the ASCE Council gives general direction to the Program: S. W. Jens (Chairman); W. C. Ackermann; J. C. Geyer; N. S. Grigg; C. F. Izzard; D. E. Jones, Jr.; and L. S. Tucker. M. B. McPherson is Program Director (23 Watson Street, Marblehead, Mass. 01945). Administrative support is provided by ASCE Headquarters in New York City.

Automatic Control

Since its inception in 1967 the ASCE Program has been monitoring leading-edge projects of local governments in an effort to stimulate state-of-the-art advances elsewhere. For example, the master plans for combined sewer overflow pollution abatement in San Francisco and Chicago,(9) which call for eventual incorporation of some form of automatic control.

Installation of new storage and treatment facilities in combined sewer systems is expected to increase the complexity of operations substantially. Automatic control is viewed by many as a means for maximizing the operational efficiency of these more complicated systems. Should storage and treatment be adopted as a means for abating pollution from separate storm sewer runoff, there will be an even greater interest in using automatic control.

Anticipation of rainfall over combined sewer systems in advance of its occurrence is a goal sought by all developers of automatic control capabilities. The exploratory study report which follows is offered as a modest contribution towards the reaching of that goal.

Acknowledgments

This report covers the introductory phase of the doctoral research of Mr. Van-Thanh-Van Nguyen on storm tracking and storm characterization, one of several components of a new Urban Water Resources Program in the Department of Civil Engineering, Ecole Polytechnique of Montreal. The other two authors were his advisors on the study. (The writer is privileged to serve the Department as a quarter-time Invited Professor under an agreement between ASCE and Ecole Polytechnique, and thus has filled two roles in connection with this report).

The ASCE Urban Water Resources Research Council is indebted to the Department of Civil Engineering of Ecole Polytechnique of Montreal for its generous contribution of this report as a public service.

Processing, duplication and distribution of this Technical Memorandum was supported by Grant No. ENV77-15668 awarded to ASCE by the U.S. National Science Foundation. However, any opinions, findings, and conclusions or recommendations expressed herein are those of the authors, including this writer, and do not necessarily reflect the views of the National Science Foundation.

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Footnote

The above references Numbers 1, 2 and 3 are being published collectively in a single volume by the U.S. EPA as Urban Runoff Control Planning, EPA-600/9-78-035, 187 pp., October, 1978.

FEASIBILITY OF STORM TRACKING FOR AUTOMATIC CONTROL OF COMBINED SEWER SYSTEMS

(Addendum 6 of the ASCE Program report "Urban Runoff Control Planning," June, 1977)

by
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November, 1978

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SECTION 1

INTRODUCTION

Synopsis

The objective of the empirical study reported herein was to explore the feasibility of tracking storms for combined sewer system automatic control applications. Data used were for the largest depth events in a twenty-seven month record of hourly rainfall from a network of eighteen gages in the Montreal region. Simplistic hypotheses have been exploited to extract the maximum amount of information from the limited data base. Despite reservations about the adequacy of the data base, it is concluded that the feasibility of tracking storms for automatic control applications is suggested by the findings. However, all that has been demonstrated is that it is possible to estimate expected rainfalls one observation interval in advance, one interval at a time. Much more work will be required, with more extensive data bases, before desirable tracking capabilities can be attained. Noted are encouraging advances that are taking place in hydrometeorological research.

To insure attribution of opinion, a narrative style has been employed in most of the text.

Background

Ecole Polytechnique is affiliated with the University of Montreal and is located on the campus of the University. Under development by the Department of Civil Engineering of Ecole Polytechnique (E.P.) is an Urban Water Resources Program to serve metropolitan Montreal. Greatly facilitating the organization of the Program is a written agreement between the Montreal Urban Community (M.U.C.) and Ecole Polytechnique for cooperation and collaboration in urban water resources research, development and training. The M.U.C. is a metropolitan multiple-purpose special district that includes among its responsibilities, mainly for the island of Montreal, the collection of all wastewaters and their treatment by the design, construction and eventual operation of the required interceptors and treatment plant. While the M.U.C. is committed to some degree of supervisory control of its interceptor system, it has felt obliged to investigate the feasibility of automatic control for later consideration. As a service to the M.U.C. in this connection, E.P. embarked on an exploratory study, reported herein, as one of several projects in its new Program.

Figure 1 shows the locations of the islands of Montreal and Laval. The City of Montreal occupies the central portion of the 180-square mile island of Montreal, the remainder being under the jurisdiction of several other cities and towns. The City of Laval occupies the 90-square miles of the island of Laval. (In a separate project, E.P. is assisting the City of Laval in precipitation data analysis). Most of the three million people in the Montreal metropolitan area live on the islands of Montreal and Laval.

Automatic Control

An observation made in 1974 with regard to urban water subsystems, such as combined sewer systems, remains valid today: "The big step to be taken for most subsystems is the third modeling step – the control model – to bring to operational fruition the ultimate: fully automatic operational control".(1) The components for the latter are graphed in Figure 2.(2) The developmental bottleneck in Figure 2 is "automatic control signals generation" (lower left box).

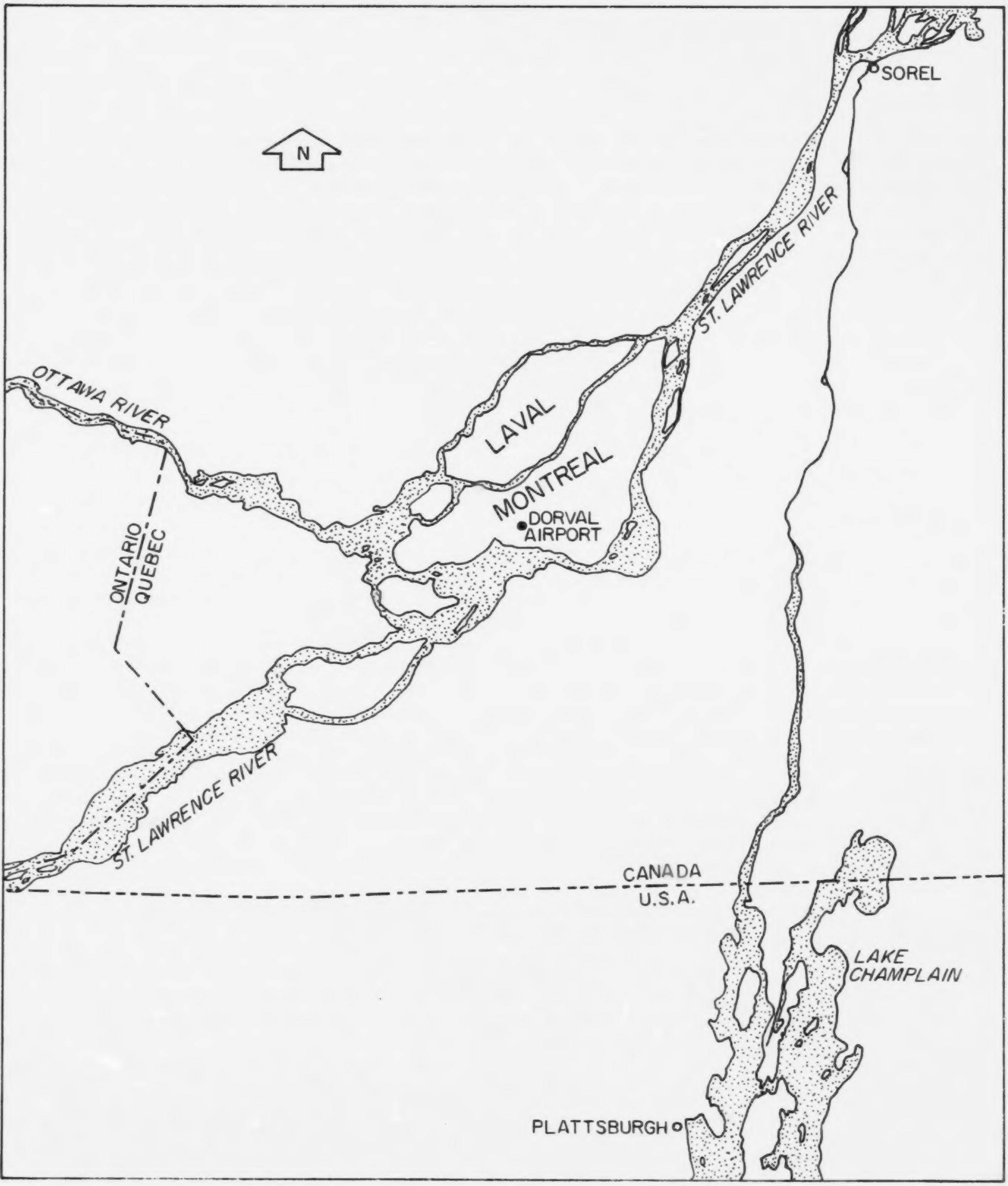
N
SOREL
ST. LAWRENCE RIVER
OTTAWA RIVER
LAVAL
MONTREAL
DORVAL AIRPORT
ONTARIO
QUEBEC
ST. LAWRENCE RIVER
CANADA
U.S.A.
LAKE CHAMPLAIN
PLATTSBURGH

FIGURE 1- MONTREAL REGION

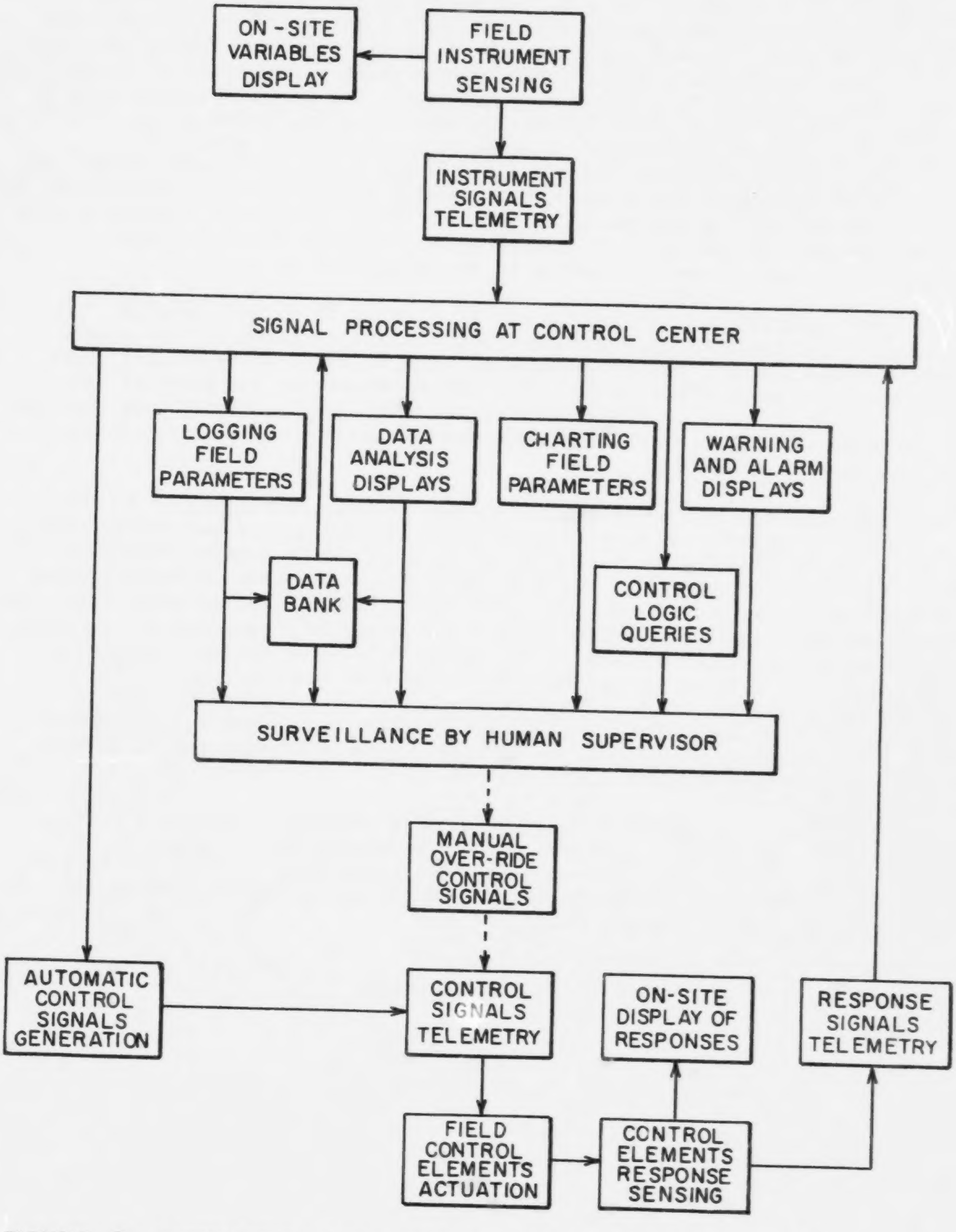
ON-SITE VARIABLES DISPLAY
FIELD INSTRUMENT SENSING
INSTRUMENT SIGNALS TELEMETRY
SIGNAL PROCESSING AT CONTROL CENTER
LOGGING FIELD PARAMETERS
DATA ANALYSIS DISPLAYS
CHARTING FIELD PARAMETERS
WARNING AND ALARM DISPLAYS
DATA BANK
CONTROL LOGIC QUERIES
SURVEILLANCE BY HUMAN SUPERVISOR
MANUAL OVER-RIDE CONTROL SIGNALS
AUTOMATIC CONTROL SIGNALS GENERATION
CONTROL SIGNALS TELEMETRY
ON-SITE DISPLAY OF RESPONSES
RESPONSE SIGNALS TELEMETRY
FIELD CONTROL ELEMENTS ACTUATION
CONTROL ELEMENTS RESPONSE SENSING

FIGURE 2 AUTOMATIC OPERATIONAL CONTROL COMPONENTS(2)

The most advanced conceptual development for total automatic control of combined sewer systems has been in connection with a plan for overflow pollution abatement being implemented by the City and County of San Francisco, California. General strategies were developed at Colorado State University(3) in a cooperative project with the City. Initial detailed efforts by the City have favored what might be called "reactive" control, whereby the combined sewer system would be manipulated in response to a storm as it occurred over the City. The ultimate scheme would be what might be termed "anticipatory" control, where the sewer system would still be manipulated according to the actual incidence of a storm over the system, but tempered by the expected rainfall at the start of the storm and in subsequent periods of time throughout the storm. This requires development of capabilities for storm tracking and for estimation of expected storm patterns on a probabilistic basis. Achievement of anticipatory control is particularly attractive where manipulation of large volumes of storage is involved.

San Francisco has embarked on the development of an anticipatory capability.(4) Storm-tracking and pattern-anticipation research is also a part of a project in a three-State sector centered about Chicago, where special radar equipment is being used in conjunction with the deployment of a network of 320 raingages.(5) In contrast to these efforts, the Ecole Polytechnique study reported herein is minuscule, being merely the introductory phase of the doctoral thesis research of the first author.

Returning to "automatic control signals generation" in Figure 2 (lower left box), it is important to realize that storm tracking and pattern anticipation is only one of the capabilities that have to be developed to achieve "hands-off," or complete, automatic control of combined sewer systems. Storm characterization models have next to be interfaced with models that simulate the important hydrologic and hydraulic behavioral characteristics of catchments, in groups and collectively, and the whole integrated with decision algorithms reflecting system performance criteria. All this must be accomplished in a real-time context, with shifts in tactics during a storm required in as short an interval as fifteen minutes. However, while storm tracking and pattern anticipation capability is only one of the above three elements, it is the one that is needed to advance from a reactive mode to an anticipatory mode.

Considerable background is contained in a report from Colorado State University,(6) which includes a discussion of the feasibility of real-time prediction of urban rainstorms and an extensive bibliography on urban rainfall variation. Also discussed are questions of sensitivity in the interfacing of storm models with system behavior models.

SECTION 2

RAINGAGE DATA

Raingage Network

Shown in Figure 3 are the locations of the eighteen recording raingages that formed the network for this study. The gage at Dorval Airport on the island of Montreal was chosen as the reference or target gage because it is at a first-order weather station. While there are five gages in the vicinity of Ottawa, only the Ottawa Airport gage was used because it is at a first-order weather station. The reason that all recording gages in the immediate vicinity of Montreal were used was that part of the research, not reported here, was involved with point versus areal rainfall in that vicinity.

Data for the network had to be obtained from three sources: two provinces and the national government. In retrospect, it would have been better if two gages had also been included that are located in Ontario just off the lower left corner of Figure 3. If there had been time, it would have been helpful to include a gage or two in New York and possibly a gage in Vermont. The U.S. border is only a little more than thirty miles below Montreal.

St. Jerome (Figure 3) is on the edge of the Laurentian Mountains and Ste. Clothilde (Figure 3) is in the foothills of the Adirondack Mountains. The broad and comparatively flat St. Lawrence Valley passes in a southwest-northeast direction between these ranges in the Montreal region.

Rainfall Record

Rainfall data from the eighteen recording gages was readily available only at an hourly interval, a rather severe limitation. Records were made available to Ecole Polytechnique on magnetic tapes.

The longest concurrent record for the eighteen gages was for the period August 1, 1972, through October 31, 1974. Therefore, our study was restricted to this twenty-seven month record.

Storm Selection

The record for the target or reference gage, Dorval Airport, was first examined in terms of the twenty-four hour periods with the largest total depths. From a total of 275 days with rain at Dorval, the about one-sixth with the largest depths were segregated. The smallest twenty-four hour total depth for these cases was 0.20-in. The reason the largest cases were emphasized was because we assumed that these would be the most challenging to an automatic control system. From these top twenty-four hour cases, forty-two distinct storms were identified, some of which carried over from one calendar day into the next.

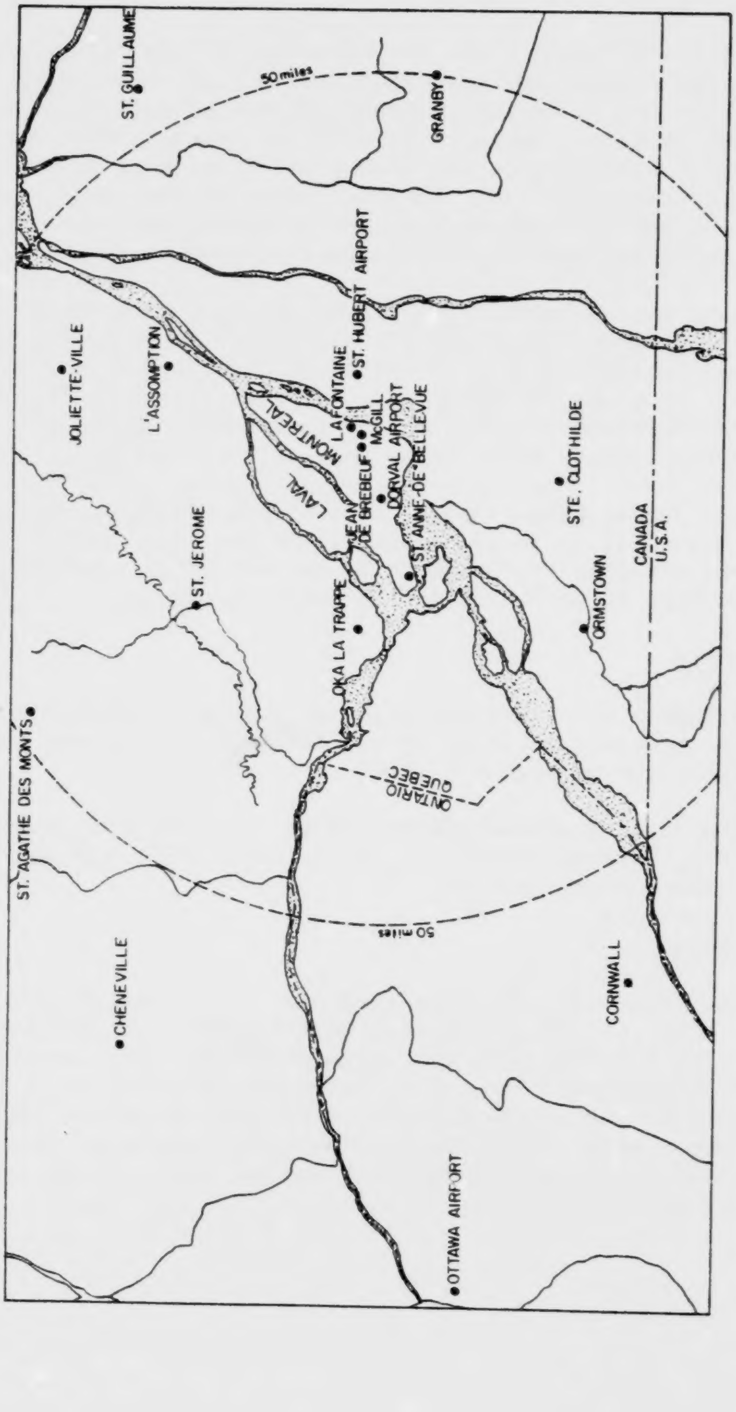
ST. GUILLAUME
50 miles
GRANBY
ST. HUBERT AIRPORT
JOLIETTE-VILLE
L'ASSOMPTION
LAFONTAINE
McGILL
DORVAL AIRPORT
ST. ANNE-DE-BELLEVUE
MONTREAL
LAVAL
JEAN DE BREBEUF
ST. JEROME
STE. CLOTHILDE
CANADA
U.S.A.
ORMSTOWN
OKA LA TRAPPE
ST. AGATHE DES MONTS
ONTARIO
QUEBEC
50 miles
CHENEVILLE
CORNWALL
OTTAWA AIRPORT

FIGURE 3 - RECORDING RAINGAGE NETWORK

We reasoned that low-intensity, long-duration cyclonic-type storms would be the easier to anticipate for automatic control applications because of the extensive continental weather forecasting that is made of frontal movements. Also, the hourly depths associated with such mild cyclonic storms are rather uniform and comparatively small, further facilitating their anticipation for automatic control. By eliminating the obvious low-intensity, long-duration cyclonic occurrences (5 cases) and those for which there were too many missing data for the network (8 cases) or there were anomalies in the data or other suspicious implications (8 cases), the original forty-two candidate storms were reduced to twenty-one.

SECTION 3

STORM ANALYSIS

Network Plotting

The records for the segregated twenty-one storms were plotted using the SYMAP(7) grid interpolation and computer plotting program. With this program, contours for a given range of values are demarcated by the use of a specific symbol for that range.

Storm Center Correlation

In the San Francisco work, a good lagged correlation had been obtained for certain hourly data between gages in the City and other gages to the north-northwest along the coast.(8,9) Inspired by these indications, from among the twenty-one storms that remained for study we identified six for which the mean storm centers appeared to pass first near the most remote gage at Ottawa Airport and then near the Dorval Airport gage. These six events had been selected by studying the network tabulated record, augmented by visual inspection of the SYMAP plots.

For the six storms, a correlation analysis was made of the Ottawa versus Dorval hourly depths for time lags of one, two, three and four hours. For time lags of two, three and four hours, the associated individual hourly depths for the Ottawa and Dorval gages were acceptably correlated, with a correlation coefficient of about 0.7 or more for five of the six candidate events. Thus, a fairly strong persistence in the occurrence of maximum depths along the mean path of the center of a storm was suggested.

Mean Storm-Center Paths

For most of the record used, the mean storm center passed away from Dorval Airport or on a path different from the Ottawa-Dorval axis. The estimated mean storm-center paths for all the twenty-one events discussed subsequently are indicated in Figure 4. These paths were selected by studying the tabulated records for the network, augmented by visual inspection of the SYMAP plots. We feel reasonably confident of the mean storm-center compass bearings indicated, but the distance by which these storm-centers passed beyond Dorval is a much more subjective estimate. However, our goal was to extract as much information as possible from the limited data available.

Our after-the-fact appraisal of storm path is obviously contrary to the need in automatic control applications to anticipate storm paths. Extensive work by the Illinois State Water Survey(5,10) has demonstrated that a special 10-cm radar augmented by perhaps a dozen telemetry-connected raingages is capable of tracking storms over a range of at least one hundred miles and of measuring rainfall amounts in a storm to an acceptable precision. We could not conceive of a control system that would be driven exclusively by raingages because of the extensive network of telemetry-connected gages that would be required and the massive amount of data processing in real time that would be involved.

We therefore postulated the position that, in future control applications, mean storm-center paths would necessarily be determined via special radars continuously calibrated against a few telemetry-connected raingages. For the

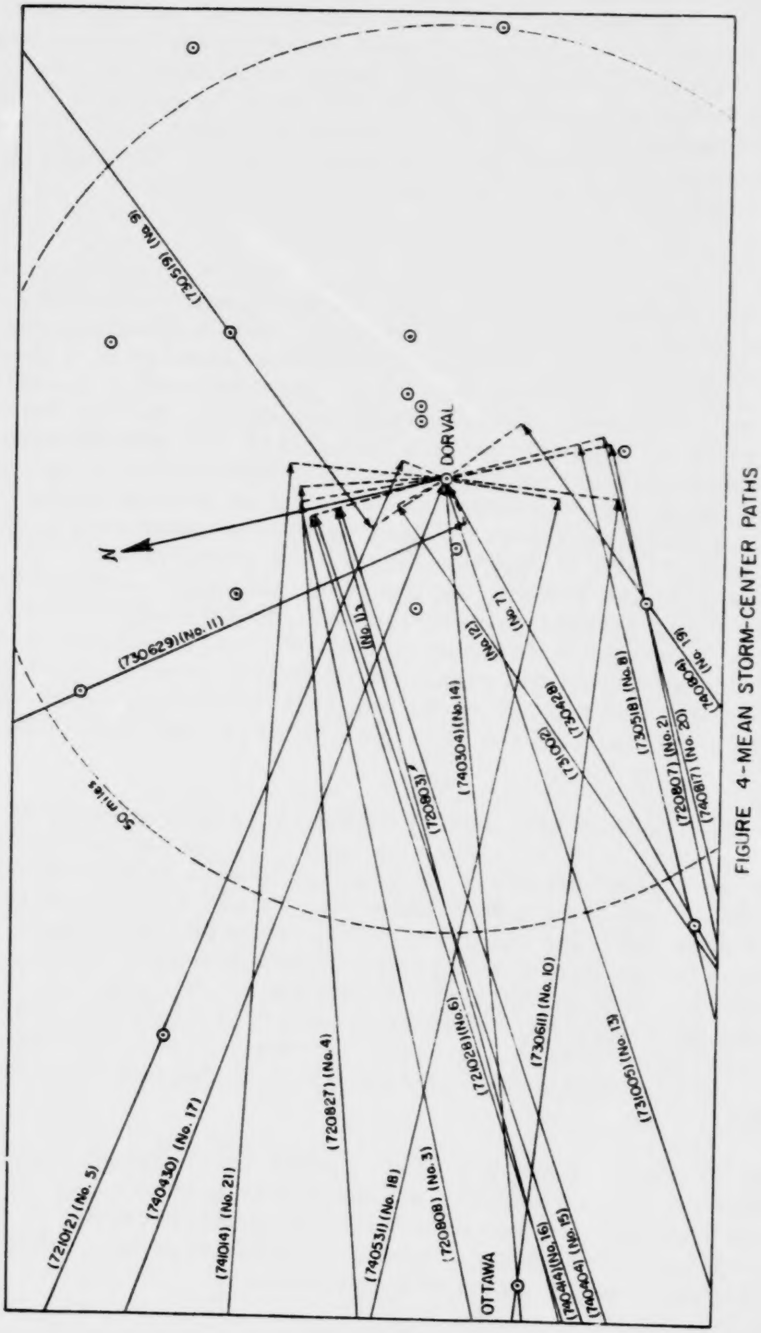
DORVAL
OTTAWA
N
50 miles
(730519) (No. 9)
(730629) (No. 11)
(721012) (No. 5)
(740430) (No. 17)
(741014) (No. 21)
(720827) (No. 4)
(740531) (No. 18)
(720808) (No. 3)
(740414) (No. 16)
(740404) (No. 15)
(721028) (No. 6)
(730611) (No. 10)
(731005) (No. 13)
(720803) (No. 1)
(740304) (No. 14)
(731002) (No. 12)
(730428) (No. 7)
(730518) (No. 8)
(720807) (No. 2)
(740817) (No. 20)
(740804) (No. 19)

FIGURE 4-MEAN STORM-CENTER PATHS

purposes of our study we somewhat arbitrarily chose a fifty mile radius from Dorval as the reference circle for the succeeding analyses. We reasoned that mean storm-center paths could be determined adequately outside this periphery with special radars. As a matter of convenient coincidence, we had found that a two-hour lag time from Ottawa to Dorval was the hourly multiple that provided the best correlation, on the average, for storm centers that passed approximately along the Ottawa-Dorval axis. Ottawa Airport is very close to one-hundred air miles from Dorval Airport. Thus, we would assume initially that the appropriate time lag from a fifty mile circle around Dorval would be one hour. We had been advised that storm movement was generally around half that speed, so we investigated a two hour and three hour lag time as well. The results of these analyses are presented in the next Section.

Reference Depths

Returning to Figure 4, only a few of the twenty-one events had estimated mean storm-center paths that passed over Dorval. Some sort of relationship had to be hypothesized for the other cases. Shown schematically in Figure 5 is a representative "mean path of center of storm," together with a parallel line passing through Dorval. Where the parallel line passing through Dorval crosses the fifty-mile circle of Figure 4 we have designated the interpolated rainfall depth as d_{50}. The observed rainfall depth at Dorval is designated as d_D. We have assumed that d_{50} and d_D are correlatable. This is an extrapolation of our meager findings for the Ottawa-Dorval axis in that we have assumed that a degree of proportionality exists between depths along a line parallel with the mean storm-center path as was inferred between depths along the mean storm-center path itself. This is to say that the lateral distribution of rainfall amounts is assumed to be similar as a storm traverses a fifty mile distance. Or, putting it another way, we assumed that not only does the magnitude of hourly depth tend to persist at the storm center as it moves along its path, but that the hourly depth along a line parallel with the mean storm-center axis also tends to persist along that slice of the storm.

Before leaving the question of storm path, some mention should be made of our interpretation of it. We were aware of the fact that convectional storms are made up of cells that grow and decay in a mostly random fashion and that cyclonic storms have pulsating bands of higher intensity rainfall. The center of mass of the aggregate of such cells or bands would thus be expected to wander rather than follow a straight line or even a continuous curve, and these tendencies could be observed even with the limited data at our disposal. However, in an engineering application, rigor must give way to simplification to arrive at applicable tools. We feel fairly comfortable about the use of a mean storm-center path as it was derived from our data. Only more extensive and detailed evidence would provide a greater assurance.

Two reviewers of a draft of this report felt that the hypothesized lateral variations in rainfall depths schematically portrayed in Figure 5 represented a too idealized or too deterministic simplification of complex processes. We are confident that succeeding studies elsewhere using more generous networks will lead to greatly improved hypotheses.

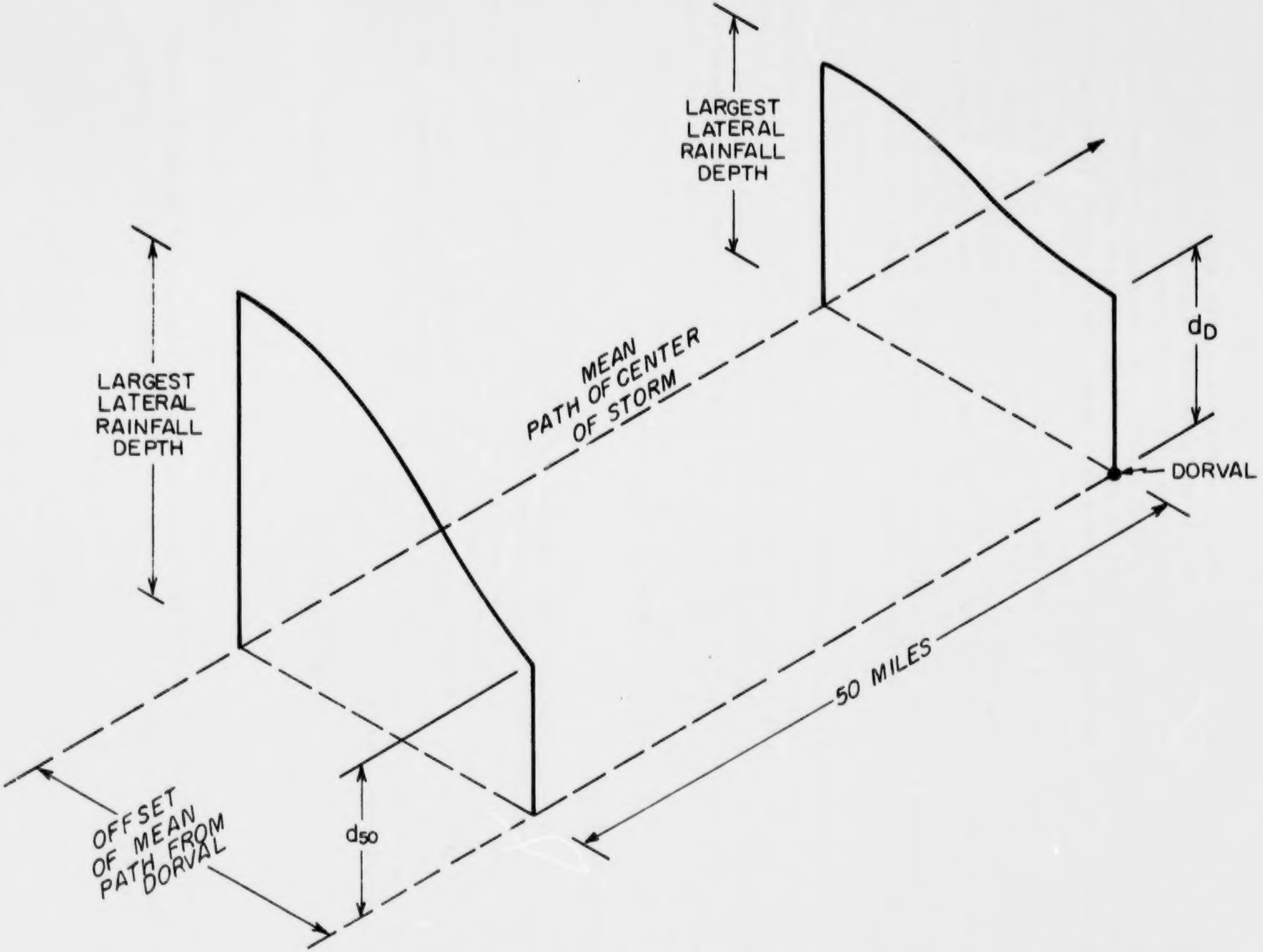
LARGEST
LATERAL
RAINFALL
DEPTH
LARGEST
LATERAL
RAINFALL
DEPTH
MEAN
PATH OF CENTER
OF STORM
d_D
DORVAL
50 MILES
OFFSET
OF MEAN
PATH FROM
DORVAL
d_{50}

FIGURE 5 - REFERENCE DEPTHS

SECTION 4

CORRELATION ANALYSIS

Storm Data Used

Because of an automatic control orientation, our attention was focused on the largest total depth events and on the greatest rainfall intensities within these events. To reduce the data processing burden we restricted analysis to the continuous portions containing the larger depths. We reasoned that small depths, such as in antecedent rainfall, could be handled with relative ease in control applications, but if included in our data sets would distort correlations because of the disproportionately large errors in small values. For example, we dismissed values of one-hundredth to five-hundredths of an inch occurring over a period of several hours before or between periods of heavy rainfall over the raingage network. Thus, our definition of an event was of a fairly continuous occurrence of larger depths. Some events continued from one calendar day into the next.

Parenthetically, in characterizing a sixty-seven year record of hourly values for a gage in San Francisco, events were defined in terms of the major rainfall portions of storms.(4)

Characteristics of the twenty-one events used in the analyses to be reported are listed in Table 1. Indicated therein are: the event total duration at Dorval, T_D; the total event depth interpolated at the reference point, D_{50}, and recorded at Dorval, D_D; and the maximum hourly depths, $(d_{50})_m$ and $(d_D)_m$, and average hourly depths, $\overline{d_{50}}$ and $\overline{d_D}$, at the two points. Hourly and other depths at the reference point (e.g., d_{50} and D_{50}) were all obtained from interpolated values as read from the SYMAP plottings. As noted earlier, the mean storm center paths for the twenty-one events are shown in Figure 4.

Regression Analyses

We had hypothesized that because a two-hour time lag had produced the best correlation, on the average, between Ottawa and Dorval hourly depths, over a distance close to 100-miles, a one-hour lag time should be appropriate in relating the interpolated hourly depths 50-miles distant from Dorval, d_{50}, with those observed at Dorval, d_D. Simple linear regression analysis of the observed hourly depths at Dorval at time t+1, d_D^{t+1}, against the interpolated hourly depths at the reference point an hour earlier, d_{50}^{t}, resulted in the following equation of fit for all twenty-one events:

$$\hat{d}_D^{t+1} = 0.7246\, d_{50}^{t} + 0.0214 \qquad \text{(Equation 1)},$$

where $\hat{d}_D^{t+1}$ is the hourly depth predicted by the equation. For Equation 1, the simple correlation coefficient was 0.6417 and the standard error of estimate was 0.0671-in.

TABLE 1 - EVENT CHARACTERISTICS

Event Number	Event Date (yr/mo/dy)	Event Duration (hr.)	Total Event Depth (in.)		Maximum Hourly Depth (in.)		Average Hourly Depth (in.)	
		T_D	D_{50}	D_D	$(d_{50})_m$	$(d_D)_m$	$\overline{d_{50}}$	$\overline{d_D}$
1	720803	6	.522	1.170	.24	.47	.087	.196
2	720807	10	1.170	.720	.26	.25	.117	.072
3	720808	10	.800	.660	.39	.34	.080	.066
4	720827	11	.685	.340	.22	.19	.062	.031
5	721012	3	.210	.210	.14	.16	.070	.070
6	721028	10	.935	.840	.25	.16	.093	.084
7	730428	4	.070	.220	.03	.08	.018	.055
8	730518	12	.580	.440	.16	.13	.048	.036
9	730519	6	.235	.350	.06	.10	.039	.058
10	730611	3	.420	.470	.25	.41	.140	.156
11	730629	5	.520	.270	.32	.16	.104	.054
12	731002	6	.300	.260	.12	.06	.050	.043
13	731005	10	1.270	1.040	.41	.36	.127	.104
14	740304	14	1.110	1.210	.16	.17	.079	.086
15	740404	5	.460	.630	.46	.23	.092	.126
16	740414	4	.210	.220	.15	.20	.052	.055
17	740430	8	.300	.430	.06	.10	.037	.054
18	740531	3	.200	.200	.11	.14	.066	.066
19	740804	5	.450	.330	.16	.18	.090	.066
20	740817	4	.245	.250	.12	.18	.061	.062
21	741014	7	.180	.300	.04	.09	.026	.043

Because we had been advised that storm movement generally required closer to two hours to traverse a distance of 50-miles, we next made a simple linear regression analysis for a two-hour time lag. The following equation of fit was obtained for all twenty-one events, for the observed hourly depths at Dorval at time t+1, d_D^{t+1}, against the interpolated hourly depths at the reference point two hours earlier, d_{50}^{t-1}:

$$\hat{d}_D^{t+1} = 0.1246\ d_{50}^{t-1} + 0.0663 \qquad \text{(Equation 2).}$$

For Equation 2 the simple correlation coefficient was 0.0122 and the standard error of estimate was 0.0870-in.

Simple linear regression analysis for a three-hour time lag resulted in a simple correlation coefficient of only 0.0021 and a standard error of estimate of 0.0874-in.

Of the three time lags tested, the one-hour time lag was obviously superior. However, several researchers have noted a degree of dependence between the hourly rainfall at a given station and the consecutive hourly rainfall at that station. In an attempt to incorporate some accounting for any serial dependence of hourly values, a multiple stepwise linear regression analysis of six variables was performed. The dependent variable was the hourly depth at Dorval, d_D^{t+1}, one hour after a given depth had occurred at the fifty-mile distant reference point, d_{50}^t. Four independent variables in addition to d_{50}^t were included among the total of six random variables: d_{50}^{t-1}, d_{50}^{t-2}, d_D^t and d_D^{t-1}. For a lack of significance, two of the independent variables were rejected, d_{50}^{t-2} and d_D^{t-1}. The resulting expression for the twenty-one events was as follows:

$$\hat{d}_D^{t+1} = 0.7788\ d_{50}^t - 0.2850\ d_{50}^{t-1} + 0.1474\ d_D^t + 0.0274 \ \text{(Equation 3).}$$

That is, with this expression, given the rainfall depth at the reference point fifty miles from Dorval at time t (d_{50}^t) and t-1 (d_{50}^{t-1}), and the depth at Dorval at time t (d_D^t), the depth at Dorval can be predicted at time t+1 ($\hat{d}_D^{t+1}$). The multiple correlation coefficient for the above equation was 0.6643 and the standard error of estimate was 0.0649-in.

Equation 3 seemingly represents only a very modest improvement over Equation 1. After comparing the standard errors of estimate for the individual events, the overall standard errors of estimate for these two equations, and the predicted versus the observed values of d_D for each of the twenty-one events, we concluded that the two equations were of about equal validity, although Equation 3 held a slight edge. We decided to use Equation 3 exclusively in the remainder of this presentation because we suspect that the inclusion of two more independent variables, d_{50}^{t+1} and d_D^t, in this equation might tend to compensate, respectively, for the variations overlooked in taking one hour as the average lag time for a slice of all the twenty-one storms as it moved from a point 50-miles distant from Dorval to Dorval, and the effects of dependence of rainfall amounts in consecutive rainy hours.

Because the lag time between the reference point and Dorval was taken as one hour for Equation 1 and is the lag time of the dominant independent variable in Equation 3, listed in Table 2 are the individual event simple correlation coefficients for the observed hourly depths at Dorval, d_D^{t+1}, versus the interpolated reference depths, d_{50}^t. In Table 2, the correlation coefficients for fifteen of the twenty-one events are 0.756 or larger. Considering the errors inherent in the crude means that were employed to estimate d_{50}, we have no basis for concluding that the interpolated values of d_{50} were more dependable for these fifteen cases, or that they were less dependable for the six cases with correlation coefficients ranging between 0.589 and 0.670.

Plotted in Figures 6 through 11 are cumulative curves for all twenty-one events of the observed rainfall at Dorval at t+1 and of the values regenerated using Equation 3, termed "fitted" in these graphs. The dispersion for each event displayed in Figures 6 through 11 has been quantified as the standard error of estimate (SE). Listed in Table 3 are the SEs for each of the twenty-one events, for values of hourly depths of Dorval that satisfy Equation 3, $\hat{d}_D^{t+1}$, versus those actually observed at Dorval, d_D^{t+1}. Because all event parameters – such as total event depth, hourly event depths, total storm duration – are of importance in automatic control applications, it is difficult to interpret the physical meaning of the relative magnitudes of the SEs.

Returning to Figures 6 through 11, we wish to draw attention to the fact that, because of the self-corrective nature of Equation 3, the errors in individual time steps are smoothed out over the total event. That is, overall, the equation does track the correspondence between d_{50}^t and d_D^{t+1}. Of course, the acceptability of errors in individual hour projections would be a function of the control system catchment size and the degree of control precision sought, as well as the state of any storage that might be involved at that time.

We are satisfied that the results described in this subsection show considerable promise for further development of storm tracking capability where more generous data bases are available.

It is important to remember that our objective was to explore the possibility of developing an anticipatory model, not to develop an operational model. Therefore, it was not necessarily appropriate for us to split the twenty-one events into calibration and verification components. Furthermore, the very small number of usable events, and the loss of supporting information from the equal number of events rejected, militated against viewing the test model as being more important than it should be regarded.

Radar Check

After the mean storm-center paths had been estimated on the basis of listed hourly depths for all eighteen raingages and reference to the SYMAP plots, radar photos were checked for further confirmation. However, radar records were available for only thirteen of the twenty-one events involved. The weather radar installation at issue is located in the western part of the island of Montreal at Ste. Anne de Bellevue, Figure 3. However, it was possible to discriminate only the mean directions of the storms but not their mean paths because gradations in intensity were not discernable on the photos. As a result of this check, the direction of two of the storms was revised somewhat. As noted earlier, the mean storm-center paths graphed in Figure 4 are the ones that were used in arriving at the values of d_{50} used in the correlation analyses.

TABLE 2

CORRELATION COEFFICIENTS FOR INDIVIDUAL EVENTS

Event Number	Event Date (yr/mo/dy)	Correlation Coefficient (d_D^{t+1} vs. d_{50}^{t})
		r
1	720803	.911
2	720807	.883
3	720808	.792
4	720827	.620
5	721012	.998
6	721028	.756
7	730428	.870
8	730518	.589
9	730519	.589
10	730611	.994
11	730629	.992
12	731002	.823
13	731005	.641
14	740304	.889
15	740404	.888
16	740414	.968
17	740430	.670
18	740531	.997
19	740804	.881
20	740817	.924
21	741014	.648

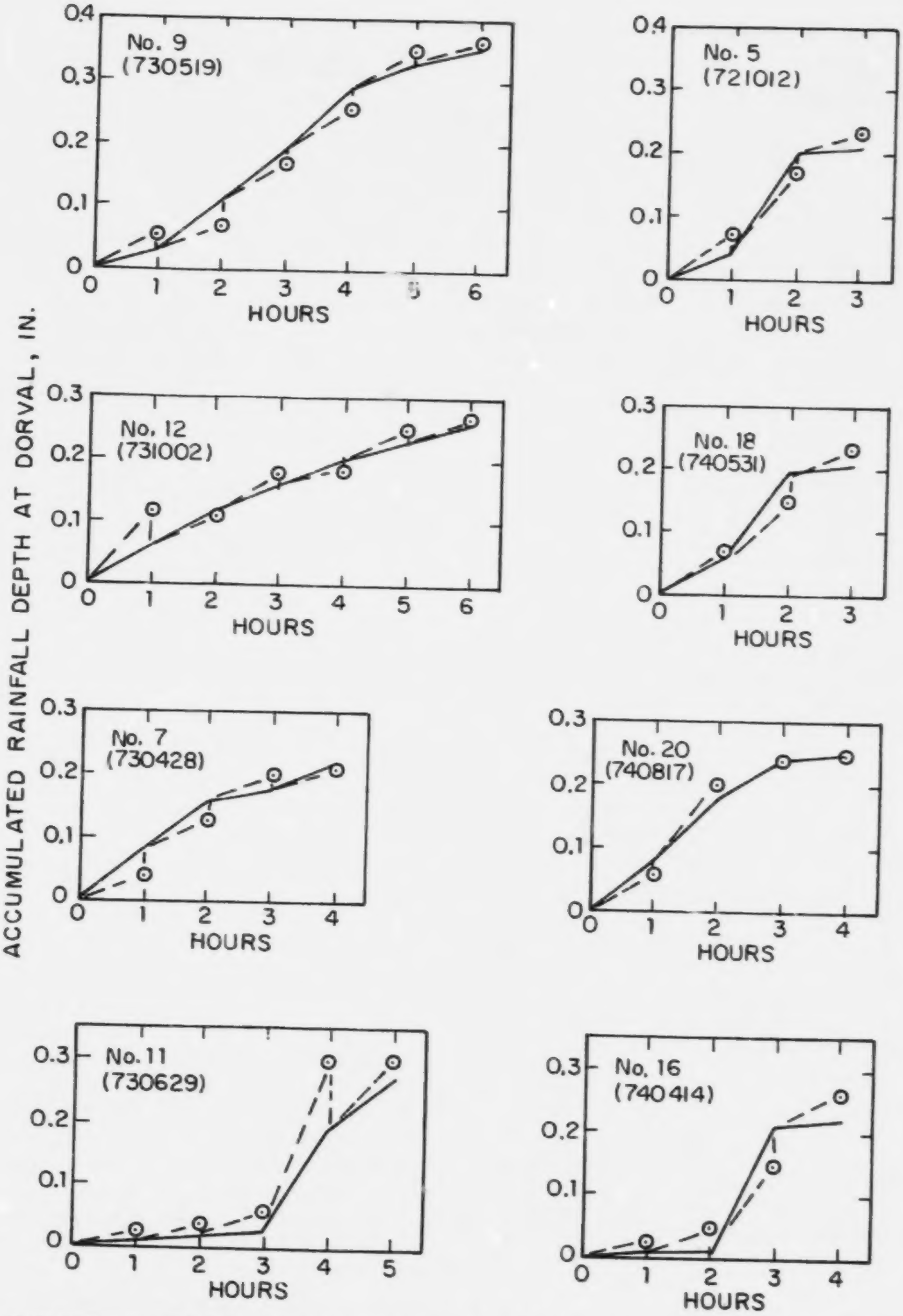

FIGURE 6- OBSERVED (——) AND FITTED (– –⊙) AT DORVAL

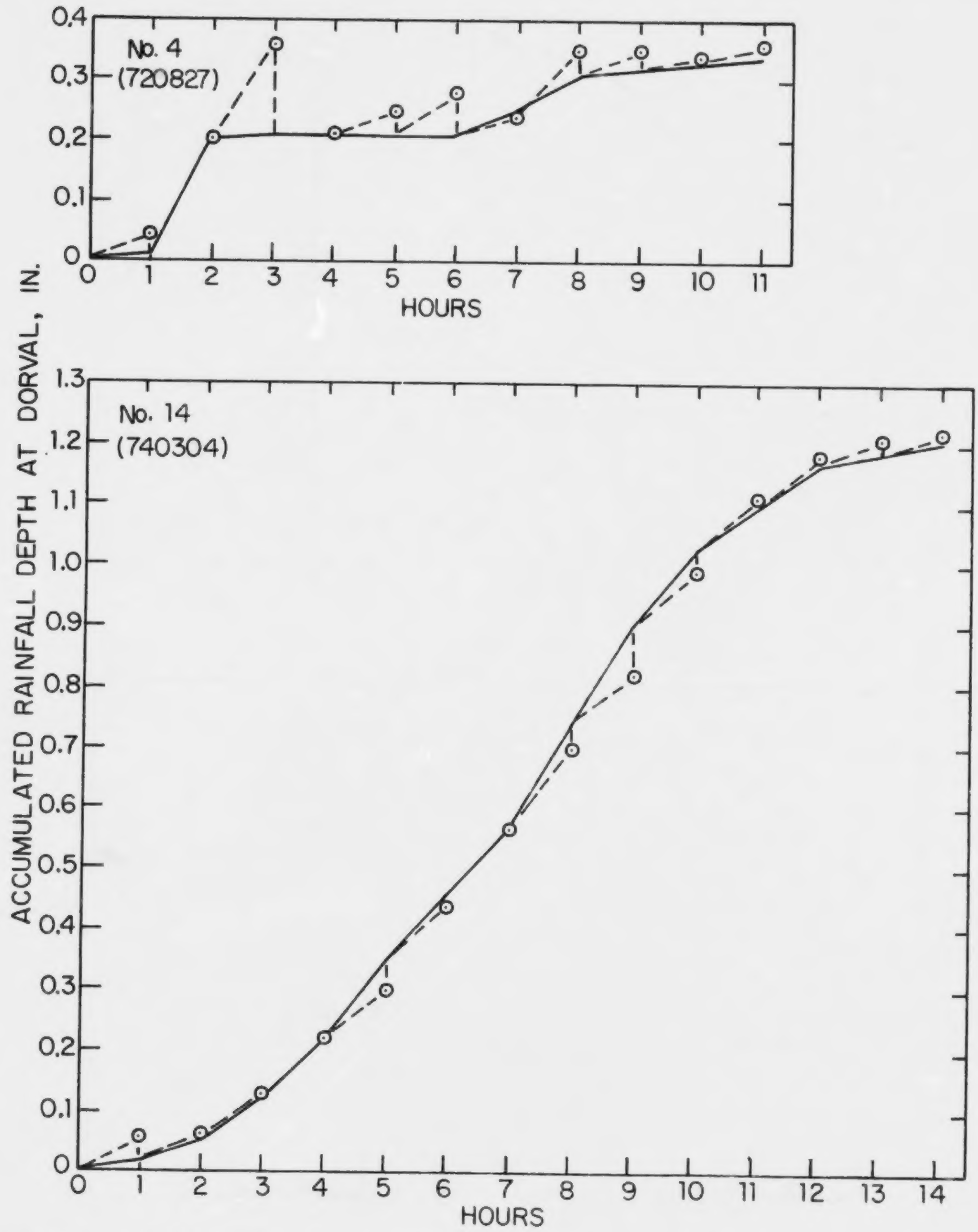

FIGURE 7- OBSERVED (—) AND FITTED (---⊙) AT DORVAL

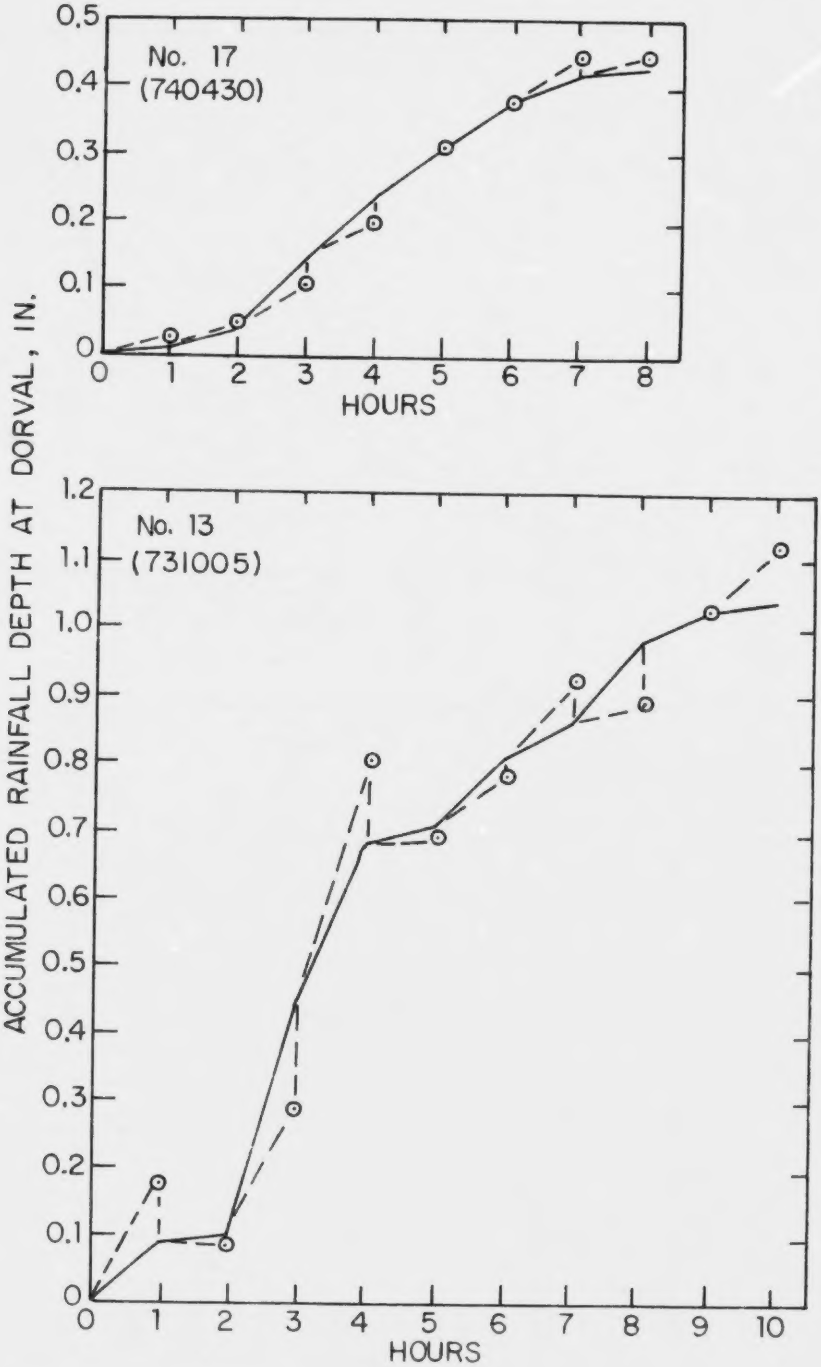

FIGURE 8- OBSERVED (—) AND FITTED (– – ⊙)
AT DORVAL

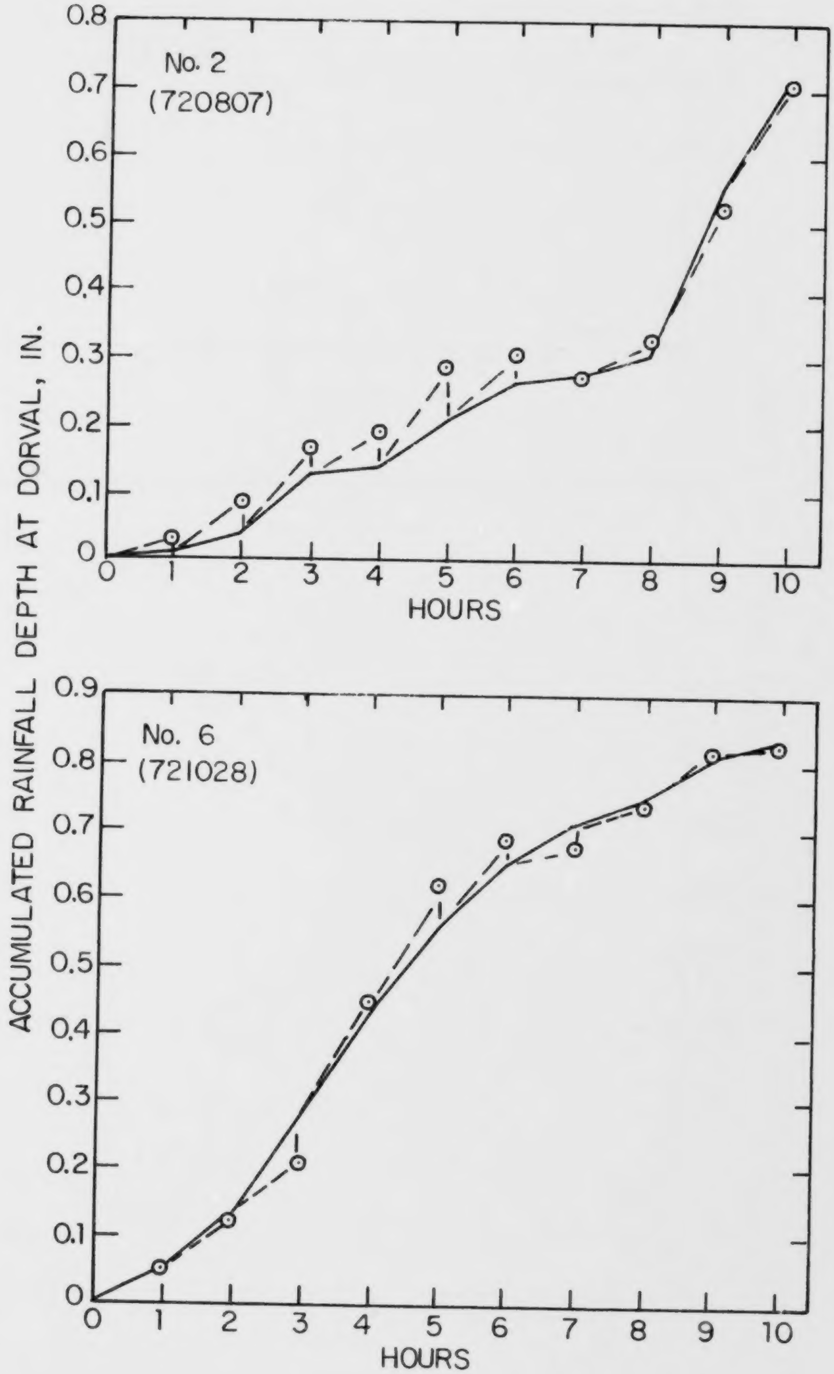

FIGURE 9- OBSERVED (——) AND FITTED (— — ⊙)
AT DORVAL

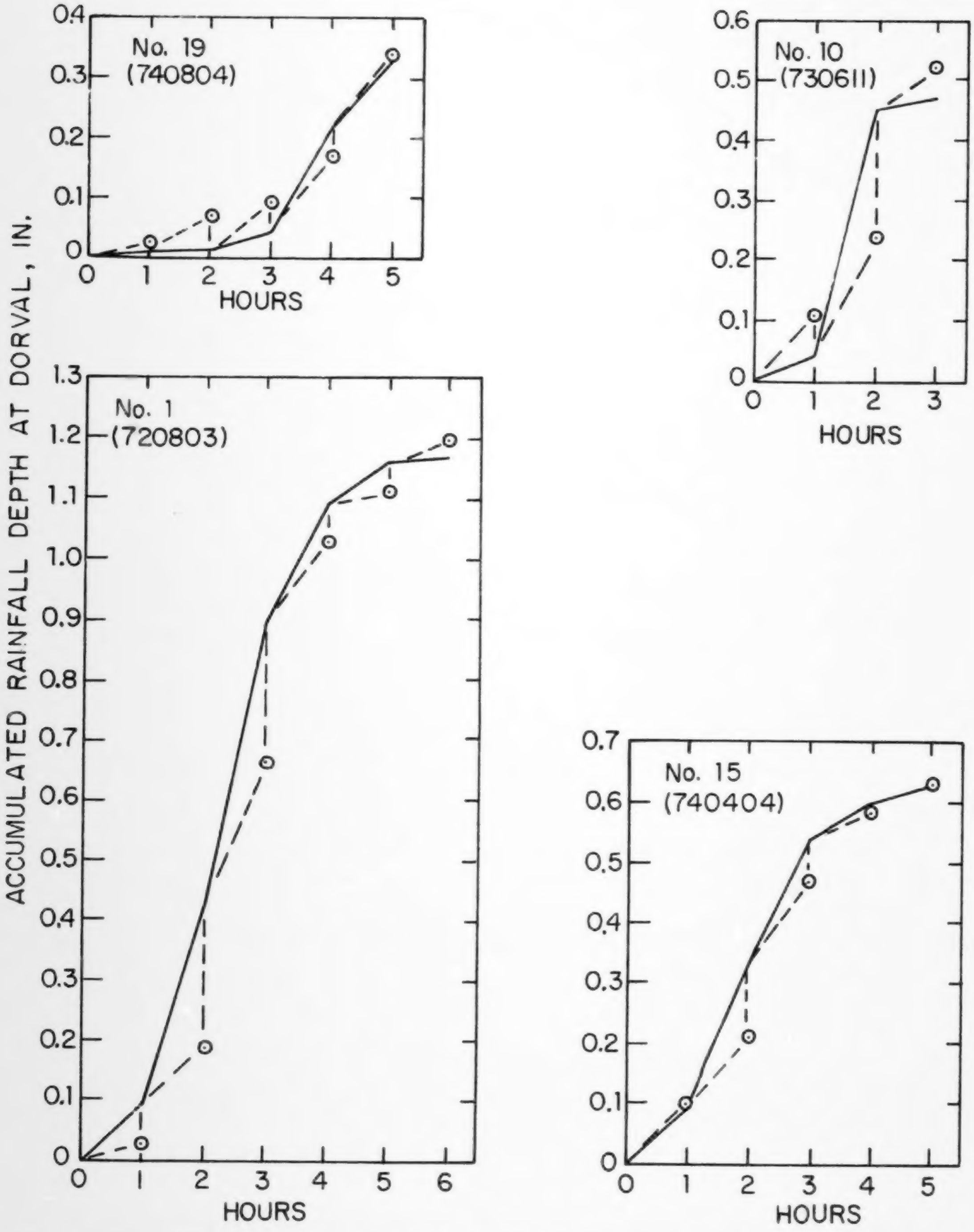

FIGURE 10- OBSERVED (—) AND FITTED (— —⊙) AT DORVAL

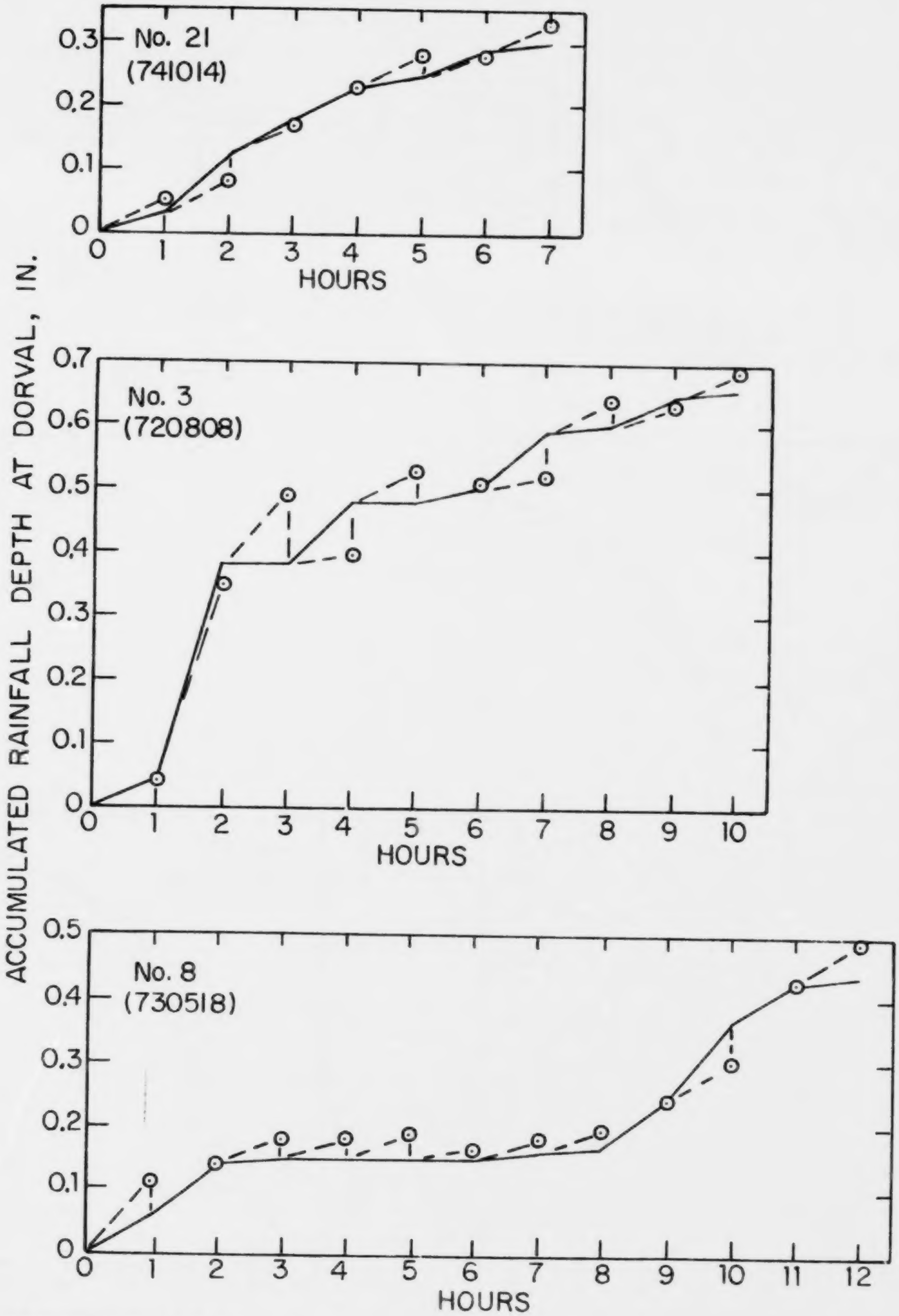

FIGURE 11- OBSERVED (—) AND FITTED (– –⊙) AT DORVAL

TABLE 3

STANDARD ERRORS FOR INDIVIDUAL EVENTS

Event Number	Event Date (yr/mo/dy)	Standard Error of Estimate, Equation 3 ($\hat{d}_D^{t+1}$ vs. d_D^{t+1})
		S.E. (in.)
1	720803	.170
2	720807	.046
3	720808	.061
4	720827	.063
5	721012	.045
6	721028	.037
7	730428	.035
8	730518	.040
9	730519	.030
10	730611	.226
11	730629	.072
12	731002	.033
13	731005	.094
14	740304	.034
15	740404	.082
16	740414	.060
17	740430	.028
18	740531	.053
19	740804	.059
20	740817	.016
21	741014	.027

Of interest in passing is that members of the Stormy Weather Group, McGill University, Montreal, had used records from a special radar at Ste. Anne de Bellevue several years ago to study summer storms that occurred between 1969 and 1972 over Ottawa, some 85 miles away.(11) Their interest centered on a study of events which had led to the flooding of house basements in Ottawa.

Parameter Exploration

An attempt was made to correlate such variables as storm compass bearing, offset of mean storm path from Dorval, total event depth, total event duration, etc. Because of the inexactness of storm path estimates and the small number of events involved, no responsible conclusions could be drawn. However, in future more extensive studies elsewhere it may be possible to arrive at the type of generalizations sought.

SECTION 5

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The objective of this study was to explore the feasibility of tracking storms for combined sewer system automatic control applications. We attempted to extract the maximum amount of information from a sparse network of recording rainfall gages for which only hourly data were readily available. As a consequence of our limited data base, we have been obliged to make simplistic hypotheses on the character and movement of storms. However, this should not be taken as a disparagement of the network, which serves other, quite different purposes.

Despite the preceding reservations, we feel that this exploratory study has suggested that the tracking of storms for automatic control applications is feasible. However, all that has been demonstrated is that it is possible to estimate expected rainfalls one observation interval in advance, one interval at a time. The ultimate in predictive tools would also estimate expected total event duration, total event depth and total event pattern. We are confident that analyses from the present and future use of special radars(12) will make it possible to predict, say, half-hourly values of rainfall on a specific sector of a jurisdiction at least two hours in advance.

With respect to storm pattern characteristics, advances continue to be made on statistical characterization of rainfall records for a single point for intervals of an hour or more.(13) Storm total durations and storm total depths seem to be highly independent of each other, as do event total durations and event total depths. The small number of event cases in Table 1 reflect this common observation. Perhaps no realistic total storm prediction tool can be developed. That is, perhaps the best that can be done will be evaluation of storms as they occur, in a real-time sense, with only about a two-hour lead time. We are nevertheless optimistic because of indications from San Francisco of a banding of total event durations and total event depths(9) and because of research under way in the U.S.(12) and the U.K.(14) where special radars are being used to develop physical and climatological models of storms over metropolitan areas for local jurisdiction drainage system applications.

To reiterate, storm tracking is only part of the capability required for automatic control. Also needed are compatable hydrologic-hydraulic system models and the incorporation of operating rules and stratagems in a computer format.(15,16) These three elements must be operable in a real-time mode to be fully effective.

Recommendations

We noted at the outset that the study reported here was merely the introductory phase of the doctoral research of the first author. His theoretical attention was devoted to statistical analysis of point and areal rainfall. In that connection, we obtained considerable insights from the storm tracking study. We are of the opinion that local government operating agencies would find the undertaking of a similar exploratory study to be very illuminating, whether or not the attainment of automatic control was a serious objective. There are some metropolitan areas in the U.S.(17) and Canada that have more extensive raingage networks than the one in the Montreal area. Data from these networks could be exploited to considerable advantage. Our experience with Montreal data has been that the storm tracking study has led to a much better grasp of rainfall data information potentialities and shortcomings for planning and design applications than would otherwise have been possible.

For the operating agency committed to the achievement of complete combined sewer system automatic control there appear to be only two options: wait for the technology to be perfected for San Francisco or Chicago or London; or invest in a suitable radar installation and commence a minimum of two years duration investigation and calibration phase to develop their own technology.

We sincerely hope that the findings from our study will provide encouragement and yet serve as a caution to those interested in automatic control of combined sewer systems. In particular, we hope that our study will help to facilitate communication between engineers and meteorologists.

Encouraging Developments

A radar installation backed with calibrating telemetry-connected raingages could also serve as part of a metropolitan flood-warning system. Flash floods are associated with thunderstorm rainfall, which occurs over fairly small areas. "Radar with a scanning radius in excess of 100 miles can provide information over data void areas. ..... The ultimate solution is 'real-time' processing and analysis of radar data by automatic computer. ..... Automatic processing techniques are currently undergoing operational testing by the NWS. A prime objective of these tests has been the improvement of hydrological forecasting techniques."(18) A metropolitan flood warning system is under development in Melbourne, Australia.(19) Feasibility of linking a weather radar to a computer in which an optimum reservoir operating program is stored, as well as a program of a hydrologic rainfall-runoff model producing the reservoir inflow hydrographs from radar-measured rainfall data, has been explored in the Federal Republic of Germany for two small adjacent river basins 13- and 22-mi.^{2} in size.(20) Further, for the two basins, synthetic hydrographs computed from radar-measured rainfall were found to be of the same accuracy as those obtained from a special dense raingage network (one recording gage per 10-mi.^{2}) set up for research purposes in the same area.(20)

Lastly, special new radars of the type alluded to earlier in this Section have been developed in conjunction with weather research of the Illinois State Water Survey.(12) A recent phase of that research has included the deployment of two radar stations and 320 raingages in a three State sector centered about Chicago, Illinois. Although the thrust of the current research is on inadvertent weather modification, in an ancillary part a radar-computer system was linked to the Waterways Control Center of the Metropolitan Sanitary District of Greater Chicago to demonstrate its possible potential for use in real-time control.(5) One of the several objectives of the research is to "demonstrate the utility of a sophisticated weather radar system in the operation of the large urban, semi-automated water resource system of Chicago and environs".(21) Also, important contributions have been made to the understanding of characteristics of severe storms associated with flash flooding.(22)

Acknowledgments

Reported in this Technical Memorandum are the findings from the introductory phase of the doctoral research of the first author. His dissertation,(23) scheduled for completion early in 1979, features a probability analysis of rainfall temporal patterns for a thirty-year record of hourly depths at Dorval Airport, and a probability analysis of the relationships between hourly point rainfalls at Dorval Airport and the associated mean rainfalls over the islands of Montreal and Laval.

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Dr. Gaston Paulin, Director of the Meteorology Department of the Ministry of Natural Resources of Quebec, made valuable comments on the meteorological aspects of the study with respect to the data used. Messrs. Constantin Mitci, Guy Paquin and Nguyen Nam of the Water Purification Department of the Montreal Urban Community made valuable comments on the methodology developed with respect to practical applications. Professor André Leclerc, Head of the Civil Engineering Department at Ecole Polytechnique, gave encouragement to the study and facilitated its administrative support.

SECTION 6

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IMPSWM

A Research and Services Project for the Implementation of Storm Water Management Modeling

by

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Presented at the

Stormwater Management Model (SWMM)
Users Group Meeting
Montreal, Quebec
May 25, 1979

IMPSWM

A RESEARCH AND SERVICES PROJECT FOR THE IMPLEMENTATION OF STORM WATER MANAGEMENT MODELLING

Storm Water Management requires new drainage alternatives and new tools for the analysis of drainage systems. Continuous efforts are required from all those involved in drainage studies and in particular from municipalities and consulting firms to keep abreast with new developments in this field.

In the last few years, the Rational Formula is being gradually replaced by various hydrologic models such as SWMM, ILLUDAS, HYMO, etc. Many municipalities and consultants have already invested time, efforts and money in acquiring various aspects of the storm water modelling technology. Modelling of zero runoff increase has become a key issue in some new developments and its results are presented to nonspecialists. Conflicting results and delays are sometimes caused by the fact that storm water management modelling is still a developing technique.

Developers and taxpayers are interested in money saving procedures, municipal engineers need simple methods with adequately safe design. One of the key problems in urban drainage design is the standardization and simplification of modelling procedures.

After consultation with many major direct and indirect model users, the Department of Civil Engineering, Faculty of Science and Engineering of the University of Ottawa, has started in February 1979, a project coordinated by Professor Paul Wisner, with the main goal of assisting efforts of various organizations in the implementation and application of practical modelling in urban drainage.

The main tasks of the program are

1) to assess the use of some existing applications of SWM models in practical problems;

2) to improve and test some parts of SWM models used by participants in the project;

3) to coordinate modelling experience between a group of interested researchers, municipal engineers and consultants in view of standardization and simplification of modelling procedures;

4) to organize and exchange of views between modellers, developers and regulatory agencies on the best use of models and in particular on how to design more economic drainage systems in new developments;

5) to organize information and training programs on modelling and storm water management procedures.

Participants in the project have access to the following services:

1) INFORMATION ON NEW STORM WATER MANAGEMENT TECHNIQUES AND MODELLING APPLICATIONS

2) PARTICIPATION IN TRAINING PROGRAMS INCLUDING A SHORT-COURSE PER YEAR, AND FORUMS FOR THE DISCUSSION OF STORM WATER MANAGEMENT ISSUES

3) DISTRIBUTION OF THE UPDATED OR IMPROVED COMPUTER PROGRAMS AND TESTING RESULTS

4) DISTRIBUTION OF REPORTS REGARDING THE PERFORMANCE OF MODELS AND APPLICABILITY FOR DIFFERENT TYPES OF ENGINEERING PROBLEMS.

By May 15, 1979, fourteen organizations including provincial government agencies, municipalities, a major conservation authority, consulting firms and a large developer have joined the project. Organizations interested in the implementation of practical modelling, reducing their direct or indirect expenses for the development of modelling capabilities, the standardization of modelling procedures and reducing the cost of drainage by adequate application of new techniques are invited to inquire regarding their participation.

A preliminary sponsoring committee was formed by representatives of organizations which have already committed financial support to the program.

Members of this committee are:

Mr. John Anderson, P.Eng.	Manager, Ottawa Branch, Gore & Storrie Limited
Mr. R. Cebryk, P.Eng.	Senior Drainage Engineer, City of Edmonton
Mr. Martin Fereday, P.Eng.	Project Manager, Genstar Development, Edmonton
Mr. W.J. Ford, P.Eng.	Associate, Cole, Sherman & Assoc. Ltd.
Mr. Ron Hicks, P.Eng.	Coordinator, Ministry of Natural Resources, Central Region
Mr. Ivan Lorant, P.Eng.	Chief Water Resources Engineer, M.M. Dillon Limited, Toronto
Mr. Craig Mather, P.Eng.	Manager, Flood Control, Metropolitan Toronto, Conservation Authority
Mr. Dipen Mukherjee, P.Eng.	Town Engineer, Town of Markham
Mr. Z. Novak, P.Eng.	Drainage Engineer, Water Modelling Group, Ontario Ministry of Environment
Mr. Alan Perks, P.Eng.	Manager, Hydrotechnical Group, Proctor & Redfern Limited, Toronto
Mr. Harold Reinthaler, P.Eng.	Project Manager, Fred Schaeffer & Associates, Toronto

Mr. George Walker, P.Eng.	President, Walker, Newby & Associates, Edmonton
Mr. Ron Wigle, P.Eng.	Manager, Hydrotechnical Division, James F. MacLaren Limited, Willowdale
Mr. F.T. Wilton, P.Eng.	Manager, McElhanney, Surveying & Engineering, Vancouver.

An advisory committee is also being formed. Based on a preliminary activity diagram, the research team has produced a first report on an update of the TRANSPORT model developed by Camp Dresser and McKee Inc. - Water Resources Engineers. This version has several features such as:

a) Improved surcharge computation. The new version no longer considers a surge tank analogy, and modified Hardy-Cross method is applied instead, thus eliminating unrealistic manhole storage.

b) Weir transfers are node to node, which improved stability.

c) Improved scheme for orifice simulation.

d) Print-out summary for junctions, which shows:

- Maximum computed depth and time of occurance
- Maximum computed surcharge
- Minimum depth below ground elevation
- Duration of surcharge.

e) Print-out summary for conduits which shows:

- Design flows
- Maximum computed flows and velocities and their time of occurance
- Ratio of maximum to design flows
- Maximum depth above inverts at conduit ends.

The updated version of the program was distributed to interested participating organizations. A second report on the applicability of the Rational Method is scheduled for July.

Work also continues on testing the TRANSPORT program for various specific problems such as the simulation of orifices and weirs, superpipes and drop shafts.

A first forum on the WRE TRANSPORT was held on March 8 and 9, 1979, with Dr. Larry Roesner from WRE, author of the new WRE - TRANSPORT. Another forum with Mr. Terstriep, author of ILLUDAS, is scheduled on May 26.

The first short course on Storm Water Management for Runoff Control in New Developments will also be held on October 16 to 18, 1979.

The course will consist of five workshops:

- Analysis of runoff control regulations.
- Design aspects of runoff control facilities by examples.
- Incorporation of runoff control in master planning.
- Traditional versus new techniques in drainage computations (methodology).
- Traditional versus new techniques in drainage computations (examples).

Each workshop will start with a general report followed by presentation and discussion of case studies. Reports will be prepared by the IMPSWM team and reviewed by members of the IMPSWM steering committee.

Two lectures and a panel will present viewpoints of regulatory agencies, municipalities and developers.

For information regarding the IMPSWM project, please contact Dr. Paul Wisner, IMPSWM Coordinator, phone (613) 231-7022 or Mr. Atef Kassem, Assistant Coordinator, phone (613) 231-3911, at the Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada K1N 9B4.

DESCRIPTION AND APPLICATION OF AN INTERACTIVE MINI-COMPUTER VERSION OF THE ILLUDAS MODEL

BY

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PAPER PRESENTED AT THE SWMM USERS GROUP MEETING
MAY 24 & 25, 1979 Montreal (Quebec)

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DESCRIPTION AND APPLICATION OF AN INTERACTIVE MINI COMPUTER VERSION OF THE ILLUDAS MODEL

1. INTRODUCTION

Computer simulation of the urban runoff process has received a great deal of attention over the last few years; in fact computer models have been developped and released at an impressive rate since the early 70's. However practically all of the now available rainfall/runoff models of interest have been developped for large computers. With the upcoming development of automatic control algorithms and with the proliferation of mini-computers and/or micro-processors in design offices there seems to be increasing interest in providing some tools that might be adapted to these small, but yet powerful computing systems.

The object of this paper is not to present yet another computer model but rather to describe the modifications and improvements introduced in an existing popular model, namely the ILLUDAS model, in order to make it compatible with a mini-computer system.

The first section of this paper deals with the description of this particular version of the ILLUDAS model (interactive mini-computer version) while the second section briefly describes the application of the model to two different catchments, namely the Malvern Catchment in Burlington (Ontario) and the Curotte-Papineau Catchment in Montreal (Quebec). Finally, a simple example presented in the Appendix illustrates the interactive features of the model.

2. DESCRIPTION OF THE MODEL

2.1 HARDWARE DESCRIPTION

The basic features of the ILLUDAS model have been described extensively by Terstriep and Stall (1974) and no attempt will be made here to present the model again.

Adapting a model to a particular computer configuration usually requires some modification of the algorithm. The importance of such modifications depends on the differences between the two computing systems. In this particular case, the original ILLUDAS model had to be rewritten completely in order to make it compatible with the mini-computer configuration available. This modified version of the ILLUDAS model, written in BASIC has been adapted to a HP 9830 16 k byte programmable calculator. The particular hardware configuration is shown in Figure 1.

The core of the system is the HP 9830 calculator. Program files are read sequentially from the internal cassette drive and loaded into the 16 k byte memory block. Matrix and string variable manipulations are handled on two external ROMs (Read Only Memory) shown in Figure 1. Once a program file has been loaded into the system, the file is executed, statement by statement, in a sequential manner. A common memory block saves data and/or results between loading operations. Data and results can also be stored or loaded from an external cassette memory. Simulation results are printed progressively on a 132-character line printer.

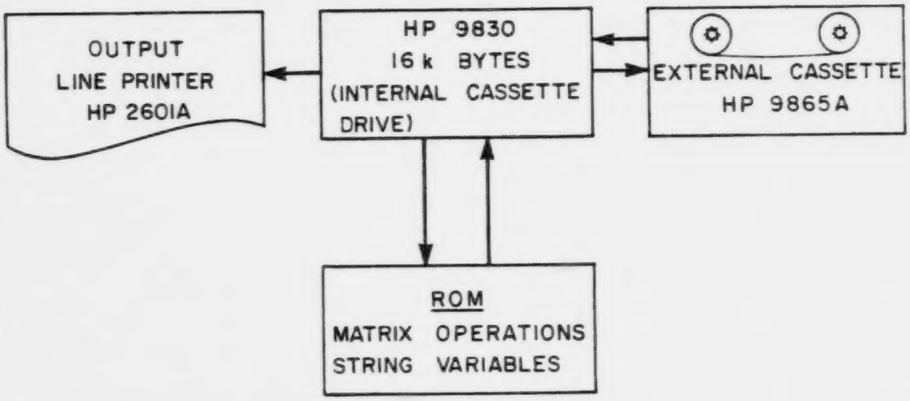
2.2 INTERACTIVE FEATURES

In rewriting into BASIC the ILLUDAS model, several modifications and improvements have been implemented and will be described more extensively in the following section. However the most significant difference of this version with the original model lies in the interactive operation of the program. While most rainfall/runoff models handle simulation in a batch-type process, this version of the model establishes a 'dialogue' with the user prior and after the simulation. It is not the intent of this paper to discuss the pros and cons of interactive computer programming, however among some of the interesting aspects of such an approach are:

- minimization of input data errors
- increase in user's confidence with the model
- accelerate the training period with the model
- excellent teaching capabilities, etc.

FIGURE 1

HARDWARE CONFIGURATION


OUTPUT
LINE PRINTER
HP 2601A
HP 9830
16k BYTES
(INTERNAL CASSETTE
DRIVE)
EXTERNAL CASSETTE
HP 9865A
ROM
MATRIX OPERATIONS
STRING VARIABLES

SYSTEM COMPONENTS

PROGRAMMABLE CALCULATOR	HP 9830A
EXTENDED MEMORY 16k BYTES	HP 11281A
MATRIX OPERATIONS ROM	HP 11270B & OPTION 270
STRING VARIABLES ROM	HP 11274B & OPTION 274
EXTERNAL CASSETTE DRIVE	HP 9865A
LINE PRINTER	HP 2607A

BESSETTE, CREVIER, PARENT, TANGUAY & ASSOCIATES

Through a visual display the program asks the user to supply the necessary information required by each subprogram block in the logical programmed sequence. Data is entered in a free-format form and syntax error are automatically pointed out in the process. The user is given the opportunity to enter or leave any of the initial sub-program blocks as he wishes prior to entering the simulation block.

In the initial stages of program execution, control is returned to the user after each block of operations through the use of the 'special function key'. As shown in Figure 2, 10 sub-program blocks are available in the initial stage of program execution prior to entering the simulation block. These subprograms are:

- "Enter catchment data"
- "Read catchment data from file"
- "Print catchment data as read"
- "Print catchment data on forms"
- "Correct catchment data"
- "Input hyetographs"
- "Print hyetographs"
- "Store catchment data"
- "Print forms"
- "Plot hydrograph from files"

Control over the program is temporarily lost after the 'Simulation' key is pressed. However, once the simulation is complete, the following options or subprogram blocks are available:

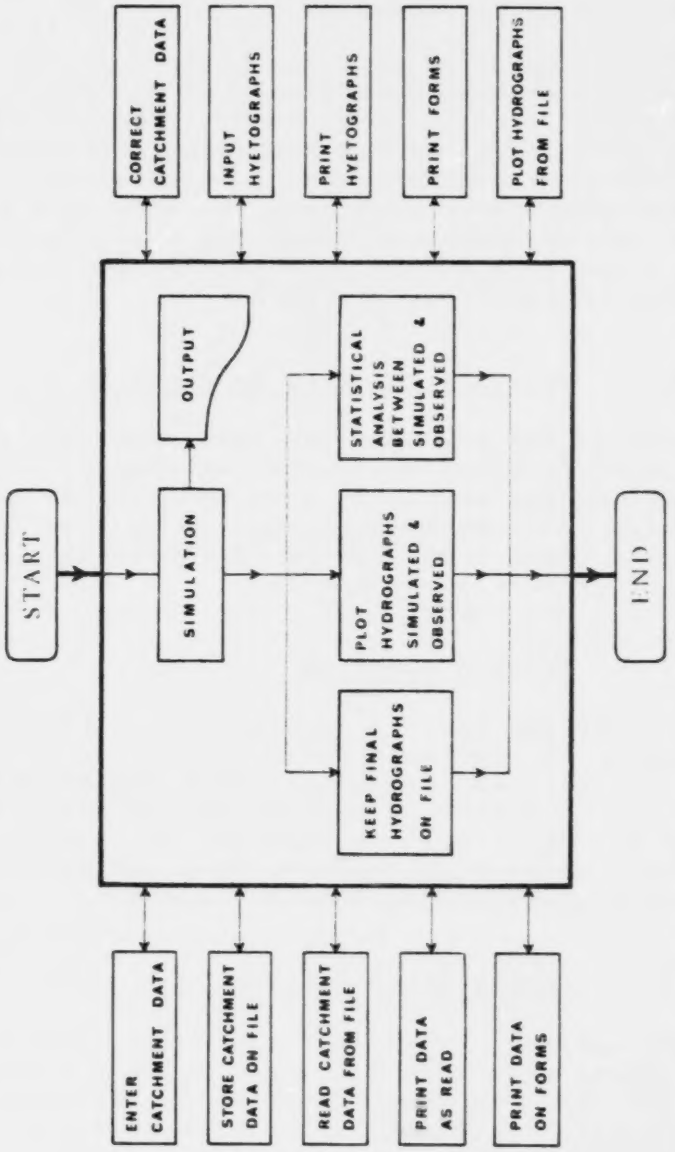
- "Store final hydrograph on file"
- "Plot hydrographs: simulated and observed (if available)
- "Statistical analysis on simulated and observed hydrographs".

2.3 PROGRAM DESCRIPTION

In adapting and rewriting into BASIC the original ILLUDAS model major programming modifications were necessary. New subroutines were created by either segmenting portions of the original model or through the addition of new features.

figure 2

INTERACTIVE FEATURES OF THE MODIFIED VERSION OF THE ILLUDAS MODEL


ENTER CATCHMENT DATA
STORE CATCHMENT DATA ON FILE
READ CATCHMENT DATA FROM FILE
PRINT DATA AS READ
PRINT DATA ON FORMS
START
SIMULATION
OUTPUT
KEEP FINAL HYDROGRAPHS ON FILE
PLOT HYDROGRAPHS SIMULATED & OBSERVED
STATISTICAL ANALYSIS BETWEEN SIMULATED & OBSERVED
END
CORRECT CATCHMENT DATA
INPUT HYETOGRAPHS
PRINT HYETOGRAPHS
PRINT FORMS
PLOT HYDROGRAPHS FROM FILE

This version of the model contains a total of 23 files (or subroutines) as shown in Figure 3, that can be loaded and executed sequentially by the system. The package now contains close to 2 000 statements.

As mentionned previously the basic features of the original ILLUDAS model have been presented elsewhere (Terstriep and Stall, 1974; Patry and Raymond, 1978) and will not be considered here. However, in rewriting the program many new features have been introduced in this particular version and are interesting to examine. It might also be of interest to point out that most of these new features have also been introduced in a recent FORTRAN (French) version of the program (Patry & Marchi, 1979).

2.3.1 Multiple rainfall hyetographs

Because of the interest and importance of spatial distribution of rainfall over large urbanized catchments, this version of the model has been designed to handle as many rainfall hyetographs as there are reaches in the catchment. In fact, every reach of the catchment can have its own specific rainfall hyetograph.

2.3.2 Inlet hydrograph

Because of the limited storage memory of the computing system it is sometimes necessary and also interesting to simulate the performance of large catchments by subdividing them into a number of smaller and more discretized subcatchments having up to a maximum of 150 reaches. This version allows generated hydrographs from upstream catchments to be fed into specific downstream branches.

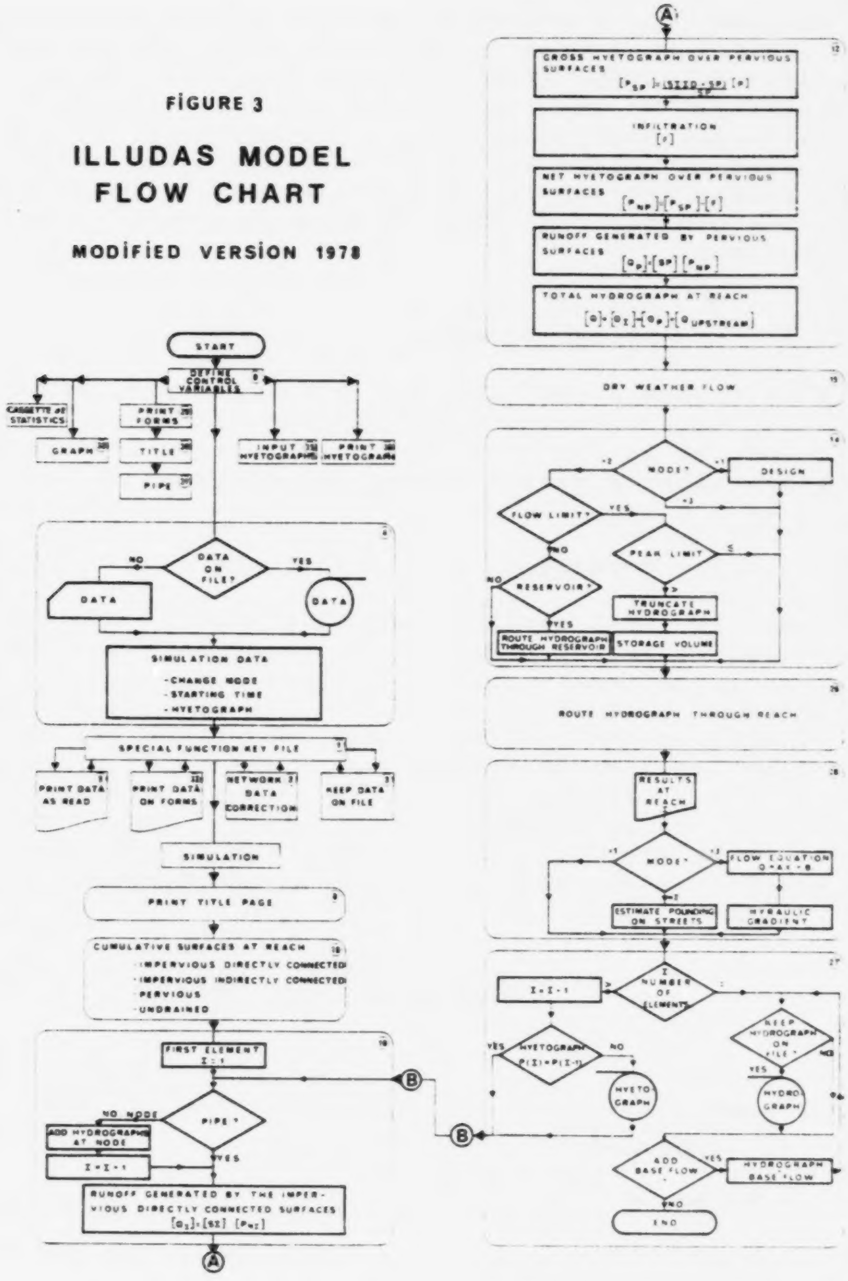
2.3.3 Infiltration parameters

In the original version of the model, Horton's soil infiltration parameters are limited to four particular soil groups. This version of the model has been designed to handle specific soil infiltration characteristics simply through the definition of Horton's infiltration parameters f_o, f_c and k.

FIGURE 3

ILLUDAS MODEL FLOW CHART

MODIFIED VERSION 1978


START
DEFINE CONTROL VARIABLES
CASSETTE of STATISTICS
GRAPH
PRINT FORMS
TITLE
PIPE
INPUT HYETOGRAPHS
PRINT HYETOGRAPHS
DATA ON FILE?
NO
YES
DATA
DATA
SIMULATION DATA
-CHANGE MODE
-STARTING TIME
-HYETOGRAPH
SPECIAL FUNCTION KEY FILE
PRINT DATA AS READ
PRINT DATA ON FORMS
NETWORK DATA CORRECTION
KEEP DATA ON FILE
SIMULATION
PRINT TITLE PAGE
CUMULATIVE SURFACES AT REACH
-IMPERVIOUS DIRECTLY CONNECTED
-IMPERVIOUS INDIRECTLY CONNECTED
-PERVIOUS
-UNDRAINED
FIRST ELEMENT I = 1
PIPE ?
NO NODE
ADD HYDROGRAPHS AT NODE
I = I + 1
YES
RUNOFF GENERATED BY THE IMPERVIOUS DIRECTLY CONNECTED SURFACES
$[Q_I] = [SI]\ [P_{NI}]$
A
B
B
A
GROSS HYETOGRAPH OVER PERVIOUS SURFACES
$[P_{SP}] = \frac{(SIID + SP)}{SP}\ [P]$
INFILTRATION
$[f]$
NET HYETOGRAPH OVER PERVIOUS SURFACES
$[P_{NP}] = [P_{SP}] - [f]$
RUNOFF GENERATED BY PERVIOUS SURFACES
$[Q_P] = [SP]\ [P_{NP}]$
TOTAL HYDROGRAPH AT REACH
$[Q] = [Q_I] + [Q_P] + [Q_{UPSTREAM}]$
12
DRY WEATHER FLOW
15
14
MODE?
=2
=1
=3
DESIGN
FLOW LIMIT?
YES
NO
PEAK LIMIT
≤
>
RESERVOIR?
NO
YES
TRUNCATE HYDROGRAPH
ROUTE HYDROGRAPH THROUGH RESERVOIR
STORAGE VOLUME
ROUTE HYDROGRAPH THROUGH REACH
RESULTS AT REACH I
MODE?
=1
=3
=2
FLOW EQUATION Q=AX+B
ESTIMATE POUNDING ON STREETS
HYRAULIC GRADIENT
I NUMBER OF ELEMENTS
>
=
I = I + 1
HYETOGRAPH P(I)=P(I-1)
YES
NO
HYETOGRAPH
KEEP HYDROGRAPH ON FILE ?
YES
NO
HYDROGRAPH
ADD BASE FLOW ?
YES
NO
HYDROGRAPH + BASE FLOW
END

2.3.4 Inlet time

Hodgson (1975) reported that one of the weaknesses of the ILLUDAS model lied in the inlet time calculation from impervious surfaces. The technique described by Terstriep and Stall (1974) was reported to generate in some instances, extremely short lag times. The kinematic wave equation was therefore introduced as an optional means to evaluate both pervious and impervious surfaces inlet times, thereby yielding a minimum time base of about 5 minutes from impervious surfaces.

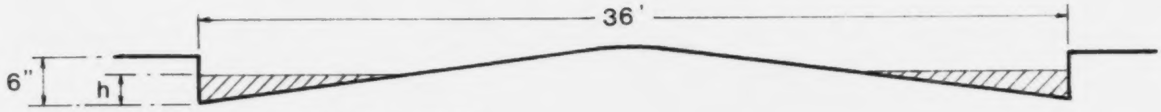
2.3.5 Design diameter

Even though traditionnally downstream pipe diameters are equal to or greater than upstream diameters, it was decided to add as an option a "hydraulic design mode" where such a constraint on pipe diameter selection is lifted.

2.3.6 Ponding on street surfaces

Under the EVAL mode, runoff in excess of the reach capacity is theoretically stored onto street surfaces and routed through the reach only when the capacity of the reach allows. Because of the difficulty in judging the importance of such street ponding, a new subroutine has been added to allow for a rough evaluation of the ponding depth on street surfaces. Ponding depths are evaluated using relationships such as those described in Figure 4. Should the ponding depth be greater than the curb height, the full right-of-way width is then taken into consideration in the flooding depth calculation.

FIGURE 4
PONDING ON STREET SURFACES



L = Maximum of
. Reach length
. 170 feet of street/acre x subcatchment area

V = Ponding volume available on street surface per acre (h max = 6 in.)
V = 9 ft^3/ft when h = 6 in.

2.3.7 Hydraulic gradeline analysis

Very often the flow that must be routed through a reach is larger than the pipe capacity. Hydrograph truncation through upstream storage is not always realistic. In order to appreciate the importance of such surcharged conditions a third simulation mode has been introduced allowing for a rough estimation of the hydraulic grade line slope necessary to carry the total peak flow through the reach. While theoretically weak this simulation mode has been found to be very helpfull in identifying problem areas within a particular catchment.

2.3.8 Base flow (dry weather flow)

In dealing with combined sewer systems it is necessary to be able to handle dry weather flow conditions. This version of the model allows for a very crude way of handling dry weather flow routing. The user must first specify the total dry weather flow to be generated by the whole catchment. This base flow is then generated automatically at each reach in direct proportion to the cumulative drained area at the reach. A much more sophisticated base flow handling routine could be implemented with little additional effort, however such a refinement has not been found necessary for our applications.

2.3.9 Statistical analysis

A statistical analysis package has been added to evaluate the goodness of fit of the simulated hydrographs when observed runoff values are available. The following statistical parameters are thus reported:

- Q_{OBS}/Q_{SIM}
- VOL_{OBS}/VOL_{SIM}
- Time to $peak_{OBS}$/Time to $peak_{SIM}$
- Required time shift to minimize R_S along with corresponding values of R, R_s and ISE.
- R : Correlation coefficient
- R_s : Special correlation coefficient coefficient
- ISE: Integral square error

2.3.10 Output and plotting features

Major changes were introduced in the presentation of the results in order to give more readily available information to the user. A sample output is shown in the Appendix.

Simulated and observed hydrographs can through the use of the plotting routine also be represented on the same graph immediately after the simulation, to allow for a quick evaluation of the results.

3. APPLICATION OF THE MODEL

This version of the model has already been applied and tested on several catchments. Results from two different size catchments will be presented here. These two catchments are the:

a) Malvern Catchment in Burlington (Ontario)

b) Curotte-Papineau Catchment in Montreal (Quebec).

3.1 MALVERN CATCHMENT, BURLINGTON (ONTARIO)

The Malvern Catchment is a typical residential, 57.6 acre catchment whose land use characteristics are described in Table 1.

A total of 12 rainfall events were selected to evaluate the performance of the model. Table 2 offers a summary of the selected events. Of the twelve events selected, seven had a maximum rainfall intensity less than or equal to 0.6 in/hr, corresponding approximately to the final minimum estimated infiltration rate from pervious surfaces. Furthermore, as reported by Marsalek (1978) the maximum runoff generated by the selected events corresponds to a once per year recurrence interval based on the simulation of historical rainfall events.

TABLE 1

CATCHMENT SURFACE CHARACTERISTICS

SURFACE SUB-TYPE	AREA				PERCENT OF CATCHMENT AREA
	IMPERVIOUS		PERVIOUS		
	acres	ha	acres	ha	
Frontyards	-	-	8.00	3.24	13.87
Backyards	-	-	30.16	12.21	52.31
Sub-total			38.16	15.45	66.18
Roofs	8.09	3.27	-	-	14.03
Streets	6.68	2.70	-	-	11.58
Driveways	3.10	1.25	-	-	5.38
Sidewalks	1.63	0.66	-	-	2.83
Sub-total	19.50	7.88	-	-	33.82
Total	19.50	7.88	38.16	15.45	100.00

TABLE 2

SELECTED RAINFALL EVENTS - MALVERN

EVENT NUMBER	DATE	STARTING TIME	DURATION	TOTAL PRECIPITATION		TOTAL PRECIPITATION 5 DAYS BEFORE		ANTECEDENT MOISTURE COEFFICIENT	TIME INTERVAL RECORDED	SIMU-LATED	MAXIMUM INTENSITY DURING ONE RECORDED TIME INTERVAL	
			(min)	(in)	(mm)	(in)	(mm)	(AMC)	(min)	(min)	(in/h)	(mm/h)
1	22 sept. 1973	6:56	138	0.71	18	0.75	19	3	2	2	2.10	53
2	23 sept. 1973	9:46	130	0.36	9	1.02	26	4	2	2	1.65	42
3	13 oct. 1973	16:14	158	0.31	8	0	0	1	2	2	0.60	15
4	28 oct. 1973	12.10	665	1.16	29	0.13	3	2	10	5	0.30	8
5	29 oct. 1973	11:20	710	1.48	38	1.66	42	4	10	5	0.48	12
6	14 nov. 1973	0:30	590	0.60	15	0	0	1	10	5	0.54	14
7	15 nov. 1973	11:30	260	0.71	18	0.64	16	3	5	5	0.36	9
8	28 nov. 1973	7:20	245	0.47	12	0.48	12	2	10	5	0.54	14
9	#4(1) 1974	19:00	41	0.63	16	1.40	36	4	1	1	3.48	88
10	#7(1) 1974	19:05	21	1.20	30	0.89	23	3	1	1	2.76	70
11	#12(1) 1974	19:06	91	0.63	16	0.08	2	2	1	1	2.03	52
12	#15(1) 1974	14:30	142	0.30	8	0.59	15	3	2	2	0.60	15
	Mean		266	0.71	18	0.65	16	3				

(1) *Rainfall supplied by J. Marsalek without date.*

3.1.1 Results

Table 3 and Figure 5 offer a summary of the simulation results using this version of the model. These results are compared in Table 4 with those obtained with SWMM (Marsalek, 1977). Figures 6 through 9 represent observed and simulated hydrographs for the first four events.

Peak flows

As shown in Table 3 the ILLUDAS model tends to underestimate the peak flow by about 11.5% on the average, with a coefficient of variation of 18.7%. Such results are similar to those reported by Marsalek (1977) using the SWMM model as shown in Table 4. While no general pattern could explain the difference between observed and simulated results, it was observed that the worst simulations were associated with the two longest rainfall events, events #4 and #5, which lasted 11.1 and 11.8 hours respectively. Careful examination of the observed rainfall/runoff data for these two events further indicated that the recorded rainfall could hardly have generated the observed flows. It has therefore been suggested that the actual rainfall over the catchment might have been different from that recorded at the gauging station (Patry & Raymond, 1979). Neglecting events #4 and #5 from the rainfall record brings the peak flow mean ($\bar{x}$) to 5.2% and the coefficient of variation to 15%.

Peak flow simulation results using SWMM are described under Table 4. While peak flows are underestimated by 7.6% on the average (using all 12 events), the coefficient of variation is slightly larger than with ILLUDAS (19.40% versus 18.7%).

Runoff volumes

Simulated runoff volumes with ILLUDAS were underestimated on the average by about 5.6% with a coefficient of variation of 7.5%. Because of the relatively low rainfall intensities, runoff from pervious surfaces did not contribute significantly to the flow for any of the events considered. Simulated runoff volumes using SWMM are on the average about 4% lower than the observed values.

TABLE 3

SUMMARY OF RUNOFF SIMULATIONS WITH ILLUDAS (AMC = 1)

EVENT NUMBER	RAINFALL DEPTH (in)	MAXIMUM RAINFALL INTENSITY (in/h)	PEAK FLOWS (CFS)			RUNOFF VOLUMES (ft^3)			TIME TO PEAKS (MIN)		
			O	I	O/I	O	I	O/I	O	I	O-I
RESULTS											
1	0.71	2.10	32.40	33.5	0.9672	54 600	48 343	1.1294	42	42	0
2	0.36	1.65	25.26	21.5	1.1749	25 400	23 770	1.0686	117	122	- 5
3	0.31	0.60	8.45	7.9	1.0696	19 900	20 288	0.9809	112	110	+ 2
4	1.16	0.30	8.52	5.4	1.5778	86 100	79 969	1.0767	316	310	+ 6
5	1.48	0.48	10.86	8.5	1.2776	110 300	102 411	1.0770	437	425	+ 12
6	0.60	0.54	10.47	9.3	1.1258	44 200	40 667	1.0869	344	355	- 11
7	0.71	0.36	6.47	6.9	0.9377	46 100	48 398	0.9525	142	145	- 3
8	0.47	0.54	9.54	8.1	1.1778	32 800	33 732	0.9724	27	35	- 8
9	0.63	3.48	31.82	37.4	0.8508	44 717	39 011	1.1463	34	30	+ 4
10	1.20	2.76	27.21	23.4	1.1628	15 925	14 023	1.1356	13	9	+ 4
11	0.63	2.04	15.11	18.5	0.8168	36 183	39 316	0.9203	13	9	+ 4
12	0.30	0.60	8.81	7.1	1.2408	20 283	17 955	1.1297	54	52	+ 2
STATISTICS											
					$\overline{x}$ = 1.1150			$\overline{x}$ = 1.0564			$\overline{x}$ = 0.5833
					S = 0.2090			S = 0.0792			S = 6.4025
					C.V. = 18.74%			C.V. = 7.50%			

O = OBSERVED
I = ILLUDAS SIMULATION

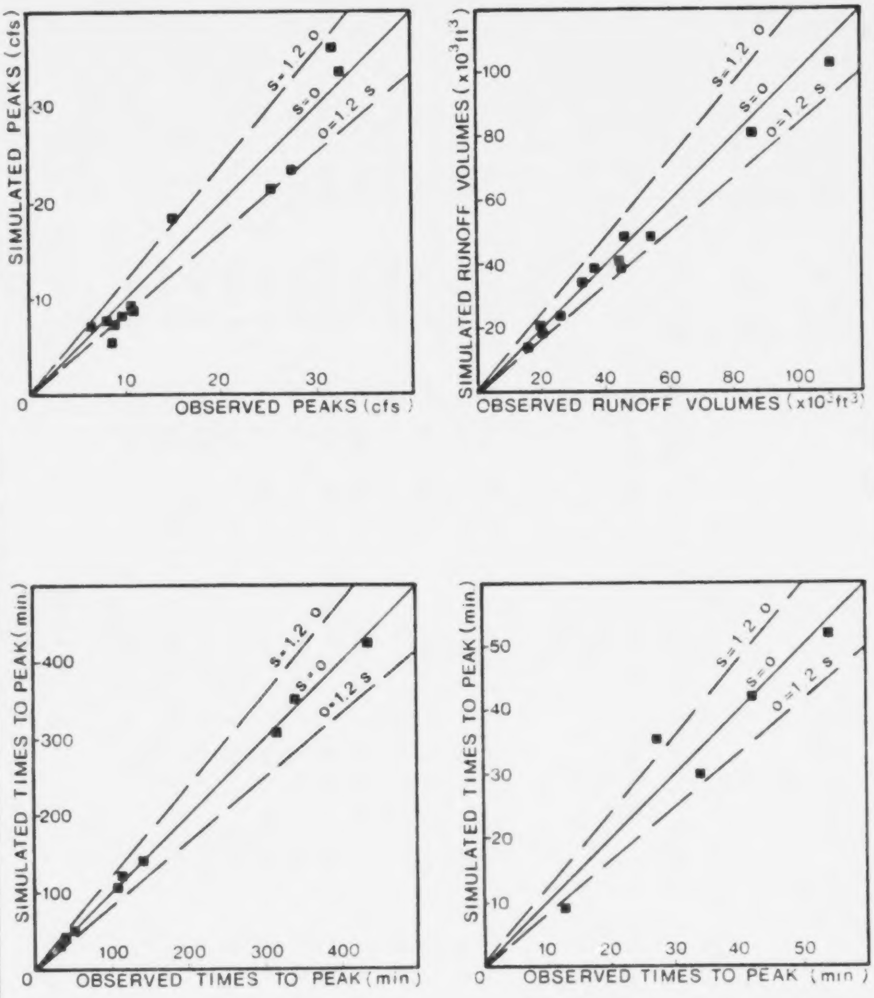


figure 5 : simulation results

TABLE 4

SUMMARY OF RUNOFF SIMULATIONS WITH SWMM (MARSALEK, 1977) *

EVENT NUMBER	RAINFALL DEPTH (in)	MAXIMUM RAINFALL INTENSITY (in/h)	PEAK FLOWS (CFS)			RUNOFF VOLUMES (ft^3)			TIME TO PEAKS (MIN)		
			O	S	O/S	O	S	O/S	O	S	O-S
RESULTS											
1	0.71	2.10	32.40	34.50	0.9391	54 600	49 500	1.1030	42	40	+ 2
2	0.36	1.65	25.26	22.40	1.1277	25 400	24 400	1.0410	117	122	- 5
3	0.31	0.60	8.45	8.30	1.0181	19 900	20 900	0.9522	112	110	+ 2
4	1.16	0.30	8.52	5.40	1.5778	86 100	80 200	1.0736	316	321	- 5
5	1.48	0.48	10.86	8.80	1.2341	110 300	103 700	1.0636	437	412	+ 25
6	0.60	0.54	10.47	9.40	1.1138	44 200	41 400	1.0676	344	354	- 10
7	0.71	0.36	6.47	6.90	0.9377	46 100	49 200	0.9370	142	148	- 6
8	0.47	0.54	9.54	9.60	0.9938	32 800	32 800	1.0000	27	44	- 17
9	0.63	3.48	31.82	38.66	0.8231	44 717	39 629	1.1284	34	28	+ 6
10	1.20	2.76	27.21	23.51	1.1574	15 925	14 418	1.1045	13	9	+ 4
11	0.63	2.04	15.11	18.55	0.8146	36 183	39 901	0.9068	13	9	+ 4
12	0.30	0.60	8.81	7.47	1.1794	20 283	18 470	1.0982	54	54	0
STATISTICS											
					$\overline{x}$ = 1.0764			$\overline{x}$ = 1.0397			$\overline{x}$ = 0.0000
					S = 0.2086			S = 0.0735			S = 10.3397
					C.V. = 19.38%			C.V. = 7.07%			

O = OBSERVED
S = SWMM SIMULATION

---

* *Several figures contained in R.R. No.57 have been corrected to reflect simulated record flows as discussed with Marsalek.*

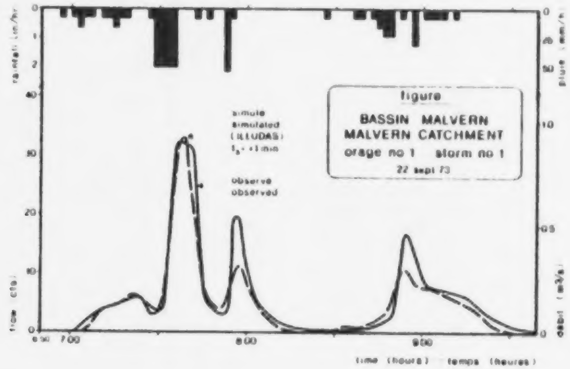


figure
BASSIN MALVERN
MALVERN CATCHMENT
orage no 1 storm no 1
22 sept 73
simule
simulated
(ILLUDAS)
t_s = +1 min
observe
observed
rainfall (in/hr)
pluie (mm/h)
flow (cfs)
debit (m³/s)
time (hours) temps (heures)
0
1
2
40
30
20
10
0
25
50
1.0
0.5
6:50
7:00
8:00
9:00

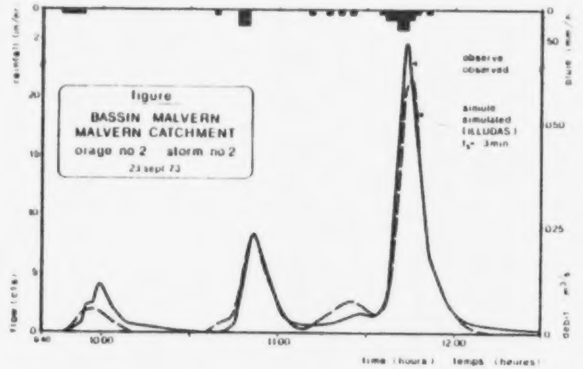


figure
BASSIN MALVERN
MALVERN CATCHMENT
orage no 2 storm no 2
23 sept 73
observe
observed
simule
simulated
(ILLUDAS)
t_s = 3 min
rainfall (in/hr)
pluie (mm/h)
flow (cfs)
debit m³/s
time (hours) temps (heures)
0
2
20
15
10
5
50
0.50
0.25
9:40
10:00
11:00
12:00

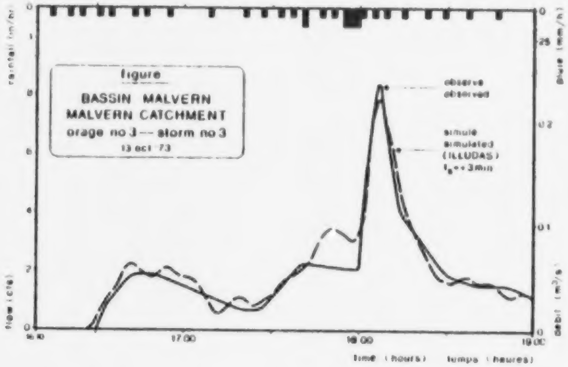


figure
BASSIN MALVERN
MALVERN CATCHMENT
orage no 3 — storm no 3
13 oct 73
observe
observed
simule
simulated
(ILLUDAS)
t_s = +3 min
rainfall (in/hr)
pluie (mm/h)
flow (cfs)
debit (m³/s)
time (hours) temps (heures)
0
1
8
6
4
2
25
0.2
0.1
16:10
17:00
18:00
19:00

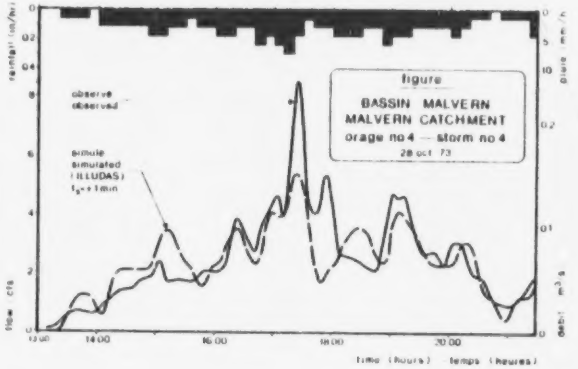


figure
BASSIN MALVERN
MALVERN CATCHMENT
orage no 4 — storm no 4
28 oct 73
observe
observed
simule
simulated
(ILLUDAS)
t_s = +1 min
rainfall (in/hr)
pluie (mm/h)
flow cfs
debit m³/s
time (hours) temps (heures)
0
0.2
0.4
8
6
4
2
5
10
0.2
0.1
13:00
14:00
16:00
18:00
20:00

More details on the results of these simulations are contained in Patry and Raymond (1979) and BCPTA (1978, 1979).

3.2 CUROTTE-PAPINEAU CATCHMENT, MONTREAL (QUEBEC)

While the Malvern Catchment was a relatively small residential catchment, the Curotte-Papineau Catchment , tributary to the new 35 mile long north shore Montreal interceptor, is a much larger urbanized catchment typical of those tributary to the interceptor. A little over 2 100 acres are drained by the combined sewer system. Land use characteristics of the catchment are described in Table 5.

TABLE 5

LAND USE CHARACTERISTICS
CUROTTE-PAPINEAU CATCHMENT
MONTREAL, (QUEBEC)

BASIC DATA	
Drainage area	2 130 acres
Population	103 000
Density	48.6 pers/acre
Imperviousness	49%
LAND USE	
Residential	59%
Commercial	20%
Other	21%

Because of our interest in using the ILLUDAS model on much larger catchments, it was decided to test and calibrate the model on this type of catchment using a total of 10 rainfall-runoff events gathered by MUC during 1975 and 1976.

The catchment was divided into 64 subcatchments having an average area of 33 acres per subcatchment. The largest single subcatchment covered over 340 acres. Table 6 offers a description of the selected rainfall events along with a summary of the ILLUDAS simulation results.

TABLE 6

CUROTTE - PAPINEAU CATCHMENT - SIMULATION RESULTS USING ILLUDAS

EVENT #	DATE	TOTAL PRECIPITATION (in)	MAXIMUM INTENSITY (in/hr)	PEAK FLOW (CFS)			VOLUME (1000 ft^3)			TIME TO PEAK		
				OBSERVED	ILLUDAS	O/I	OBSERVED	ILLUDAS	O/I	OBSERVED	ILLUDAS	O-I
1.	75/7/27	0.62	0.64	430.8	400.1	1.0766	2 880.6	3 150.0	0.9145	130	110	20
2.	75/9/2	0.88	1.98	541.7	723.2	0.7491	3 914.3	4 411.6	0.8873	50	60	-10
3.	75/10/13	0.58	0.40	317.1	297.4	1.0663	2 400.9	3 086.5	0.7779	140	145	-5
4.	76/7/16	0.84	1.91	610.9	650.0	0.9398	3 805.8	3 773.7	1.0085	80	65	15
5.	76/7/24	0.36	0.96	345.4	332.3	1.0393	1 646.2	1 804.6	0.9122	95	75	20
6.	76/5/11	0.38	0.23	203.4	186.8	1.0887	3 041.6	3 069.6	0.9909	150	145	5
7.	76/6/11	0.44	1.18	374.3	391.4	0.9564	2 832.9	3 002.4	0.9435	45	40	5
8.	76/7/27	0.81	2.32	759.2	763.1	0.9949	5 069.3	4 312.5	1.1755	50	45	5
9.	76/9/18	0.48	0.90	512.0	433.9	1.1799	2 231.2	2 317.8	0.9627	40	45	-5
10.	76/9/23	0.14	0.19	132.1	125.5	1.0523	1 197.4	1 205.0	0.9937	135	130	5
					$\bar{x}$ S C.V.	= 1.0143 = 0.1161 =11.44%		$\bar{x}$ S C.V.	= 0.9567 = 0.1021 =10.67%		$\bar{x}$ S	= 5.5 = 10.39

O = Observed Values
I = Simulated ILLUDAS values

As seen from Table 7, peak flows are particularly well simulated being underestimated on the average by only 1.4%, with a coefficient of variation of 11.4%. Such results are particularly encouraging since they tend to show the consistency of the model for different type events and different size subcatchments. It is also interesting to note that while this catchment is about 20 times less discretized than the Malvern Catchment (33 acres per subcatchment versus 1.44 acres per subcatchment) peak flow simulation results are still very good. Even though peak flow recurrence intervals have not been estimated, it might be of interest to note that flows on three occasions were larger than 50% of the full pipe capacity of the main collector and on one occasion even reached 75% of the pipe capacity estimated at 1 000 cfs.

TABLE 7

CUROTTE-PAPINEAU CATCHMENT

64 SUBCATCHMENTS

Peak Flows $\bar{x} = \frac{Q_{OBS}}{Q_{SIM}} = 1.0143$

$s = 0.1161$

$CV = 11.44\%$

Volumes of Runoff $\bar{x} = \frac{VOL_{OBS}}{VOL_{SIM}} = 0.9567$

$s = 0.1021$

$CV = 10.67\%$

Time to Peak $\bar{x} = OBS-SIM = 5.5$ min

$s = 10.4$ min

Simulated runoff volumes are on the average overestimated by about 5%, with a coefficient of variation of 10.7%. Such a difference might be associated with the differences in simulated and observed dry weather flows. Figures 10 and 11 show observed and simulated hydrographs for events #6 and #8 using the FORTRAN plotting option (Patry and Marchi, 1979).

Because of our interest in minimizing the number of subcatchments to be simulated this 2 100 acre catchment was also simulated using a single subcatchment representation, i.e. a totally lumped catchment. Results, while not reported completely herein were also very encouraging:

Peak flows	Volumes
$\bar{x}$ = 1.0046	$\bar{x}$ = 0.9198
s = 0.1505	s = 0.1121
CV = 15.02%	CV = 12.18%

4. EXAMPLE ON THE USE OF THE INTERACTIVE VERSION OF THE ILLUDAS MODEL

Appendix I shows a typical output in "printall" mode, illustrating a few features of the interactive version of the ILLUDAS model.

5. CONCLUSION

While most rainfall/runoff programs have been developped for large computers it is possible to adapt such models to small micro processors and/or mini computers. These small, but yet powerful computing systems are becoming increasingly popular in design offices, thereby creating a need for adapted rainfall/runoff models. The ILLUDAS model has been adapted to a HP 9830 - 16 k byte programmable calculator without loss in flexibility. In fact, many new features were implemented in this new version of the model.

The model has been successfully applied to two different size catchments:

a) Malvern catchment (57 acres), Burlington (Ontario)

b) Curotte-Papineau catchment (2 130 acres), Montreal (Quebec).

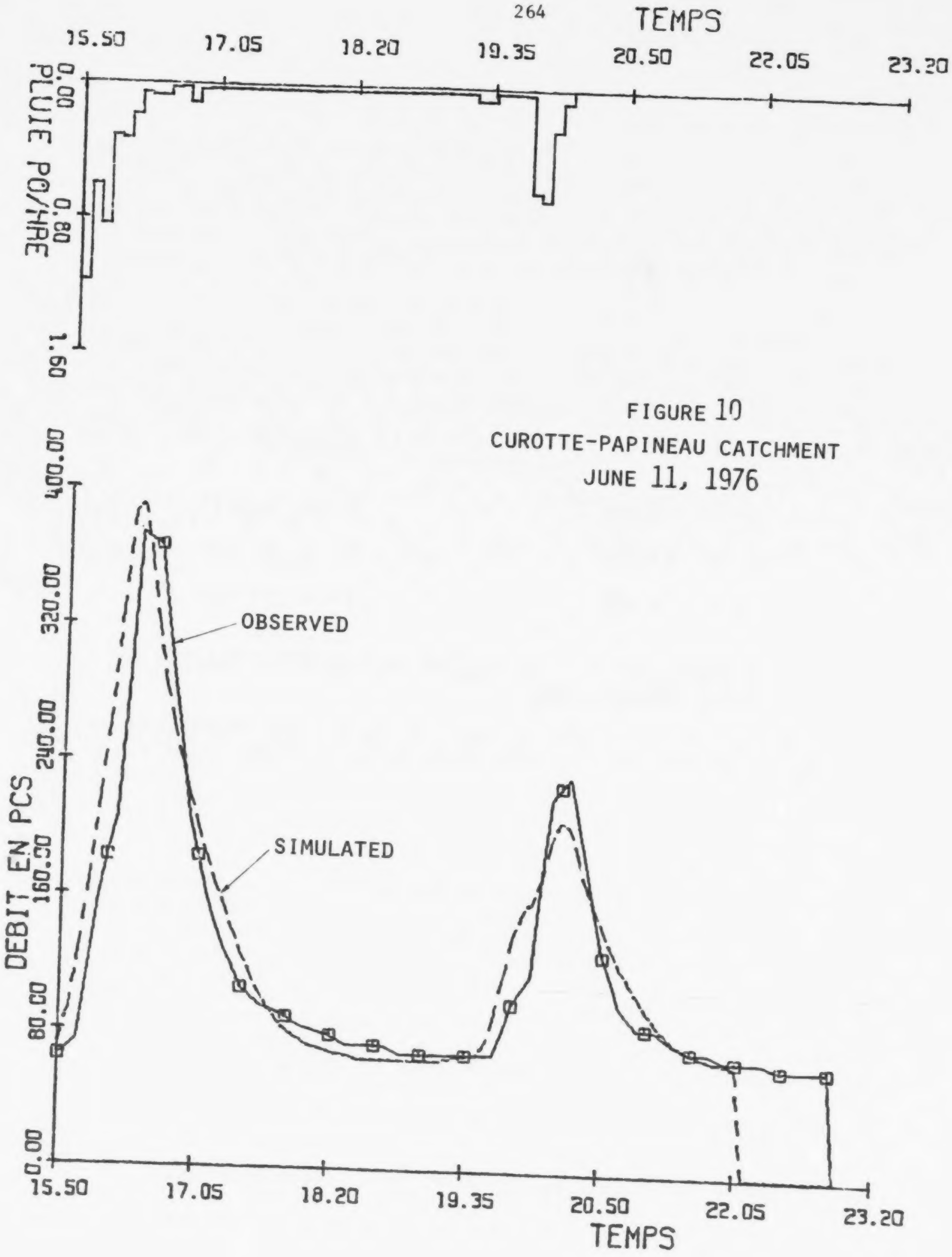


FIGURE 10
CUROTTE-PAPINEAU CATCHMENT
JUNE 11, 1976

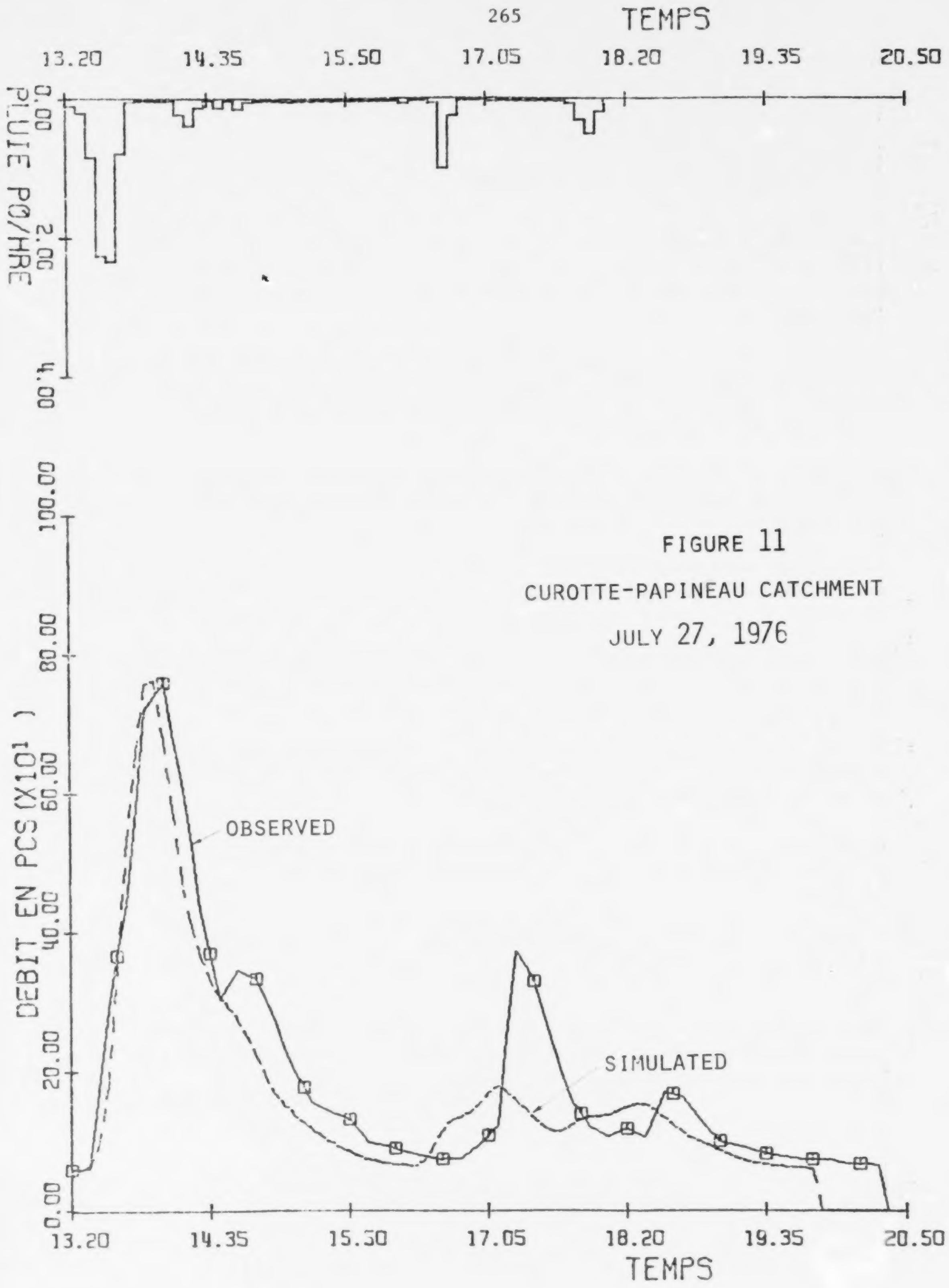


FIGURE 11

CUROTTE-PAPINEAU CATCHMENT

JULY 27, 1976

REFERENCES

1. Bessette, Crevier, Parent, Tanguay & Associates (1978) - "*ILLUDAS Model Study*", Ministry of Fisheries and Environment, Contract No 1-SS-78-00049, Progress Report No 1, October.

2. Bessette, Crevier, Parent, Tanguay & Associates (1979) - "*ILLUDAS Model Study*", Ministry of Fisheries and Environment, Contract No 1-SS-78-00049, Progress Reports No 2 and 3, March.

3. Hodgson, J. (1975) - "*A comparison of Three Runoff Simulation Models*", Greater Vancouver Sewerage and Drainage District, February.

4. Patry, G. and Raymond, L. (1978) - "*Simulation du ruissellement en milieu urbain, le modèle ILLUDAS: Présentation du modèle*", Eau du Québec, Vol. 11, No 4, Novembre.

5. Patry, G, and Raymond, L. (1979) - "*Version modifiée du modèle ILLUDAS appliquée au bassin Malvern*", Eau du Québec, Vol. 12, No 2, April.

6. Patry, G. and Marchi, G. (1979) - "*Version modifiée du modèle ILLUDAS, Manuel d'utilisateur*", in preparation, to be released in August 1979.

7. Marsalek, J. (1977) - "*Malvern Test Catchment, Volume 1*", Canada-Ontario Agreement on Great Lakes Water Quality, *RR-57*.

8. Marsalek, J. (1978) - "*Research on the Design Storm Concept*", ASCE Urban Water Resources Research Program, Technical Memorandum No 33, June.

9. Terstriep, M.L. and Stall, J.B. (1974) - "*The Illinois Urban Drainage Area Simulator: ILLUDAS*", Illinois State Water Survey, Bulletin 58, Urbana.

APPENDIX A

EXAMPLE - ON THE USE OF THE INTERACTIVE VERSION OF THE ILLUDAS MODEL

```
-----------------------------------------------------------------------------------

            BESSETTE, CREVIER, PARENT, TANGUAY & ASSOCIES
                       CONSULTING ENGINEERS

                              ILLUDAS
                              -------
                                                       PAGE...OF...
   PREPARED BY:...........                             DATE:.......

-----------------------------------------------------------------------------------
   TITLE: ...........................................................................
-----------------------------------------------------------------------------------
                          IF DATA NOT ON FILE
   A)  CONTRACT NUMBER:...........

   B)  CATCHMENT AREA:.......................ACRES
       MINIMUM DIAMETER IN DESIGN:..........INCHES
       MANNING COEFFICIENT IN DESIGN:............

   C)  DEPRESSION STORAGE ON IMPERVIOUS AREA:.......INCH
       DEPRESSION STORAGE ON PERVIOUS AREA:.........INCH

   D)  ENTRY TIME  ILLUDAS=0 K.W.E.=1 ......
       IF K.W.E.  MANNING N IMP......,N PER.......
       IF ILLUDAS MANNING N IMP..................

   E)  TIME INTERVAL......................MINUTES
       RAINFALL DURATION..................MINUTES
       RAINFALL FREQUENCY:..................YEARS
       ANTECEDENT MOISTURE COEFFICIENT (AMC 1, 2, 3 OR 4).....

   F)  DESIGN: HYDRAULIC=0 STATE OF ART=1 ......

   G)  ROUTING OPTION (0 OR 1) .....(IN DESIGN MODE)

   H)  CHANGE ONE INFILTRATION CURVE (YES OR NO) ......
       IF YES:SOIL GROUP:............
              MAXIMUM RATE:......IN/HR
              MINIMUM RATE:......IN/HR
              DECAY CONSTANT......./HR

   I)  TOTAL DRY WEATHER FLOW ........CFS

   J)  NUMBER OF PIPES + JUNCTIONS ...........

   INPUT NETWORK DATA
-----------------------------------------------------------------------------------
   IF DATA ON FILE
               IN 9865A FILE NUMBER .......
               TIME STEP, DURATION, FREQUENCY, AMC
               ......MIN,.......MIN,....YEARS,......
               TOTAL AREA .........ACRES
-----------------------------------------------------------------------------------
```

BESSETTE, CREVIER, PARENT, TANGUAY & ASSOCIATES FM2
CONSULTING ENGINEERS

ILLUDAS

PAGE...OF ...

PREPARED BY:............ DATE:........

SIMULATION IDENTIFICATION

A) PRINT HYDROGRAPHS (YES OR NO) ......

B) CITY OF ....(15)

C) CATCHMENT DESCRIPTION...............................
....(60)

D) TOTAL MODE CHANGE (YES OR NO)......
IF YES NEW MODE OR ALL PIPES ......

E) STARTING TIME .... (I.E. 06:56)

F) HYETOGRAPH (0,1 OR 2) .....
= 0 TO INPUT HYETOGRAPH
= 1 IF SPECIFIED AT EACH PIPE
= 2 SINGLE ON FILE

F) IF SINGLE ON FILE(= 2)
IN 9865A FILE NO ......

IF TO INPUT (= 0)
NUMBER OF RAINFALL STEPS .....
HYETOGRAPH (IN.)
.0.00,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....,.....,
.....,.....,.....,.....,.....,.....,.....,.....,.....

```
--------------------------------------------------------------------------------
            BESSETTE, CREVIER, PARENT, TANGUAY & ASSOCIATES
                         CONSULTING ENGINEERS

                               ILLUDAS
                               -------

  CONTRACT:...........
                                                          PAGE...OF...
  PREPARED BY:..............
                                                          DATE........
--------------------------------------------------------------------------------
  REACH NUMBER:.............
--------------------------------------------------------------------------------
  BRANCH  IREACH  I TERMI-   I CONTI-    I DES =1 I HYETO-  I SOILI AREA OF
  NUMBER  INUMBER I NATED    I NUING     I EVAL=2 I GRAPH   I 1,2 I  SUBCAT-
          I       I BRANCH   I BRANCH    I GRAD=3 I NUMBER  I 3,4 I CHMENT
--------------------------------------------------------------------------------
          I       I          I           I        I         I     I
          I       I          I           I        I         I     I
          I       I          I           I        I         I     I
--------------------------------------------------------------------------------
  LENGTH  ISLOPE  I    N     I DIAMETER IQ ALLOW  I STORAGEI
 OF FEET  I  %    I MANNING  I  INCHES  I  CFS    I 1000F↑3I
--------------------------------------------------------------------------------
          I       I          I          I         I        I
          I       I          I          I         I        I
          I       I          I          I         I        I
--------------------------------------------------------------------------------
 IMPERVIOUS AREAS
--------------------------------------------------------------------------------
    DIRECTLY     I  SUPPLEMENTAL         I
    CONNECTED    I                       I            ENTRY TIME
--------------------------------------------------------------------------------
  ACRES   I  %   I  ACRES  I    %        I MINUTESI LENGTH FEET   I SLOPE %
--------------------------------------------------------------------------------
          I      I         I             I        I               I
          I      I         I             I        I               I
          I      I         I             I        I               I
--------------------------------------------------------------------------------
 PERVIOUS AREA
--------------------------------------------------------------------------------

--------------------------------------------------------------------------------
                                         I            ENTRY TIME
  ACRES   I  %   I
                                         I MINUTESI LENGTH FEET   I SLOPE %
--------------------------------------------------------------------------------
          I      I                       I        I               I
          I      I                       I        I               I
          I      I                       I        I               I
--------------------------------------------------------------------------------
```

PROGRAM RUNNING IN PRINTALL FORM

```
LOAD0
RUN
SIMU=0 FORMS=1 GRAPH=2 HYETO=3?0
DATA ON FILE?0
 CONTRACT NO?3
AREA,MIN.DIA.,MANNING'S N?57.65,12,.013
DEPRES.STORAGE IMP & PERV ?.02,.184
ILLUDAS(0),R.W.E. (1)?0
MANNING'S FOR IMP. SURFACES?.02
TIME STEP,DURA.,FREQ,AMC?2,138,-1,1
 HYDR DESIGN (0):STATE OF ART (1)?0
ROUTING 0 OR 1?0
CHANGE INFILTRATION CURVE?1
INPUT SOIL # ,MAX,MIN,DECAY RATES?4,3,.52,4.14
TOTAL DRY WEATHER FLOW (CFS)?25
# PIPES + JUNCTIONS?6
INPUT DATA LINE 1 FOR PIPE 1     ?1,0,0,0,2,0,4,31.59
INPUT DATA LINE 2 FOR PIPE 1     ?161,.9,.013,27,0,0
INPUT DATA LINE 3 FOR PIPE 1     ?10.86,0,.99,0,9.1,0,0
INPUT DATA LINE 4 FOR PIPE 1     ?4.13,0,0,35,3
INLET HYDROGRAPH?0
INPUT DATA LINE 1 FOR PIPE 2     ?1,1,0,0,2,0,4,1.07
INPUT DATA LINE 2 FOR PIPE 2     ?396,.5,.013,30,0,0
INPUT DATA LINE 3 FOR PIPE 2     ?.49,0,.05,0,0,329,3
INPUT DATA LINE 4 FOR PIPE 2     ?.11,0,0,35,3
INPUT DATA LINE 1 FOR PIPE 3     ?2,0,0,0,2,0,4,16.09
INPUT DATA LINE 2 FOR PIPE 3     ?194,.2,.013,27,0,0
INPUT DATA LINE 3 FOR PIPE 3     ?4.43,0,.4,0,9.7,0,0
INPUT DATA LINE 4 FOR PIPE 3     ?2.35,0,0,35,3
INPUT DATA LINE 1 FOR PIPE 4     ?2,1,0,0,2,0,4,7.18
INPUT DATA LINE 2 FOR PIPE 4     ?280,.42,.013,27,0,0
INPUT DATA LINE 3 FOR PIPE 4     ?1.52,0,.14,0,5.2,0,
?0
INPUT DATA LINE 4 FOR PIPE 4     ?1.16,0,0,35,3
INPUT DATA LINE 1 FOR PIPE 5     ?0,0,2,1,0,0,0,0
INPUT DATA LINE 1 FOR PIPE 6     ?1,2,0,0,2,0,4,1.72
INPUT DATA LINE 2 FOR PIPE 6     ?176,.86,.013,33,0,0
INPUT DATA LINE 3 FOR PIPE 6     ?.56,0,.06,0,0,248,3
INPUT DATA LINE 4 FOR PIPE 6     ?.23,0,0,35,3
PRINT HYDROGRAPHS?1
CITY OF ...?BURLINGTON
CATCHMENT DESCRIPTION?MALVERN TEST BASE FLOW
TOTAL MODE CHANGE?0
STARTING TIME XX:XX ?05:56
HY.INPUT=0,MULT HY=1,SINGLE HY=2?0
NUMBER OF RAINFALL STEPS?99
 1      TH STEP?...
 2      TH STEP?..
 3      TH STEP?...
 .      .   .
 .      .   .
 .      .
 N      TH STEP?...
```

```
                                     CITY  OF BURLINGTON

                                     STORMWATER NETWORK

                                       CONTRACT 3

        MALVERN TEST WITH DRY WEATHER FLOW

        CATCHMENT AREA : 57.65   ACRES

        TIME STEP : 1.999998   MINUTES


        RAINFALL DURATION : 138  MINUTES

        ANTECEDENT MOISTURE COEFFICIENT : 1

        DEPRESSION STORAGE (IMPERVIOUS) : 0.02    INCH

        DEPRESSION STORAGE (PERVIOUS) : 0.184   INCH

        MANNING'S COEFICIENT (DESIGN) : 0.013

        HYDRAULIC DESIGN (0) ; DOWSTREAM DIA >= UPSTREAM DIA (1) : 0

        ROUTING OPTION : 0

        INFILTRATION CURVE PARAMETERS CHANGED (SOIL GROUP : 4     ) :
             MAX  RATE  = 3    IN/H
             MIN  RATE  = 0.52    IN/H
             DECAY RATE = 4.14    /H

        TOTAL DRY WEATHER FLOW: 25   CFS


        RAINFALL IN INCHES  HYETOGRAPH NO: 0
       5.56  0.000     5.58  0.000     6.00  0.000     6.02  0.000     6.04  0.000
       6.06  0.000     6.08  0.000     6.10  0.000     6.12  0.000     6.14  0.000
       6.16  0.000     6.18  0.000     6.20  0.000     6.22  0.000     6.24  0.000
       6.26  0.000     6.28  0.000     6.30  0.000     6.32  0.000     6.34  0.000
       6.36  0.000     6.38  0.000     6.40  0.000     6.42  0.000     6.44  0.000
       6.46  0.000     6.48  0.000     6.50  0.000     6.52  0.000     6.54  0.000
       6.56  0.000     6.58  0.010     7.00  0.000     7.02  0.010     7.04  0.020
       7.06  0.010     7.08  0.010     7.10  0.000     7.12  0.010     7.14  0.010
       7.16  0.020     7.18  0.010     7.20  0.010     7.22  0.000     7.24  0.000
       7.26  0.000     7.28  0.010     7.30  0.065     7.32  0.065     7.34  0.065
       7.36  0.065     7.38  0.000     7.40  0.000     7.42  0.000     7.44  0.010
       7.46  0.000     7.48  0.010     7.50  0.000     7.52  0.070     7.54  0.010
       7.56  0.000     7.58  0.000     8.00  0.000     8.02  0.000     8.04  0.000
       8.06  0.000     8.08  0.000     8.10  0.000     8.12  0.000     8.14  0.000
       8.16  0.000     8.18  0.000     8.20  0.000     8.22  0.000     8.24  0.000
       8.26  0.000     8.28  0.010     8.30  0.000     8.32  0.000     8.34  0.000
       8.36  0.000     8.38  0.010     8.40  0.010     8.42  0.000     8.44  0.010
       8.46  0.020     8.48  0.030     8.50  0.030     8.52  0.000     8.54  0.010
       8.56  0.000     8.58  0.040     9.00  0.000     9.02  0.010     9.04  0.010
       9.06  0.010     9.08  0.010     9.10  0.000     9.12  0.010
        TOTAL RAINFALL 0.71    INCHES
  HYETOGRAPH CORRECTION?0
```

```
PIPE NO 1    - 0        SOIL: 4      MODE: 2       [illegible]
*****************

TRIBUTARY AREA IN ACRES

                                 TO PIPE               CUMULATIVE
IMPERVIOUS D.C.                    10.86                    10.86
IMPERVIOUS I.C                      0.99                     0.99
PERVIOUS                            4.13                     4.13
NOT DRAINED                        15.61                    15.61
TOTAL                              31.59                    31.59

IMPERVIOUS AREA INLET TIME=   9.1MINUTES (INPUT)

IMPERVIOUS AREA HYDROGRAPH IN CFS
5.56     0.0     5.58     0.0     6.00     0.0     6.02     0.0     6.04     0.0
6.06     0.0     6.08     0.0     6.10     0.0     6.12     0.0     6.14     0.0
6.16     0.0     6.18     0.0     6.20     0.0     6.22     0.0     6.24     0.0
6.26     0.0     6.28     0.0     6.30     0.0     6.32     0.0     6.34     0.0
6.36     0.0     6.38     0.0     6.40     0.0     6.42     0.0     6.44     0.0
6.46     0.0     6.48     0.0     6.50     0.0     6.52     0.0     6.54     0.0
6.56     0.0     6.58     0.0     7.00     0.0     7.02     0.0     7.04     1.4
7.06     2.1     7.08     2.9     7.10     2.9     7.12     2.9     7.14     2.5
7.16     3.3     7.18     3.6     7.20     4.0     7.22     3.3     7.24     2.2
7.26     1.1     7.28     1.1     7.30     5.4     7.32    10.0     7.34    14.7
7.36    19.0     7.38    16.5     7.40    11.9     7.42     7.2     7.44     3.3
7.46     0.7     7.48     1.4     7.50     1.4     7.52     6.1     7.54     6.4
7.56     6.1     7.58     5.7     8.00     3.5     8.02     0.4     8.04     0.0
8.06     0.0     8.08     0.0     8.10     0.0     8.12     0.0     8.14     0.0
8.16     0.0     8.18     0.0     8.20     0.0     8.22     0.0     8.24     0.0
8.26     0.0     8.28     0.7     8.30     0.7     8.32     0.7     8.34     0.7
8.36     0.4     8.38     0.7     8.40     1.4     8.42     1.4     8.44     2.1
8.46     3.3     8.48     4.7     8.50     6.4     8.52     6.1     8.54     5.8
8.56     4.0     8.58     4.8     9.00     3.6     9.02     4.0     9.04     4.3
9.06     3.7     9.08     2.9     9.10     2.5     9.12     2.5     9.14     1.8
9.16     1.1     9.18     0.7     9.20     0.4

PERVIOUS AREA INLET TIME =  22.7MINUTES (ILLUDAS)

PERVIOUS AREA HYDROGRAPH IN CFS
5.56     0.0     5.58     0.0     6.00     0.0     6.02     0.0     6.04     0.0
6.06     0.0     6.08     0.0     6.10     0.0     6.12     0.0     6.14     0.0
6.16     0.0     6.18     0.0     6.20     0.0     6.22     0.0     6.24     0.0
6.26     0.0     6.28     0.0     6.30     0.0     6.32     0.0     6.34     0.0
6.36     0.0     6.38     0.0     6.40     0.0     6.42     0.0     6.44     0.0
6.46     0.0     6.48     0.0     6.50     0.0     6.52     0.0     6.54     0.0
6.56     0.0     6.58     0.0     7.00     0.0     7.02     0.0     7.04     0.0
7.06     0.0     7.08     0.0     7.10     0.0     7.12     0.0     7.14     0.0
7.16     0.0     7.18     0.0     7.20     0.0     7.22     0.0     7.24     0.0
7.26     0.0     7.28     0.0     7.30     0.0     7.32     0.0     7.34     0.0
7.36     0.2     7.38     0.2     7.40     0.2     7.42     0.2     7.44     0.2
7.46     0.2     7.48     0.2     7.50     0.2     7.52     0.2     7.54     0.2
7.56     0.2     7.58     0.1     8.00     0.0     8.02     0.0     8.04     0.0
8.06     0.0     8.08     0.0     8.10     0.0     8.12     0.0     8.14     0.0


DRY WEATHER FLOW AT PIPE: 14.5378457     CFS
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 ROUTED HYDROGRAPH THROUGH : 27   INCH
5.56    10.4     5.58    16.9     6.00    13.2     6.02    15.3     6.04    14.1
6.06    14.8     6.08    14.4     6.10    14.6     6.12    14.5     6.14    14.6
6.16    14.5     6.18    14.5     6.20    14.5     6.22    14.5     6.24    14.5
6.26    14.5     6.28    14.5     6.30    14.5     6.32    14.5     6.34    14.5
6.36    14.5     6.38    14.5     6.40    14.5     6.42    14.5     6.44    14.5
6.46    14.5     6.48    14.5     6.50    14.5     6.52    14.5     6.54    14.5
6.56    14.5     6.58    14.5     7.00    14.5     7.02    14.5     7.04    15.7
7.06    16.7     7.08    17.2     7.10    17.5     7.12    17.4     7.14    17.2
7.16    17.6     7.18    18.2     7.20    18.4     7.22    18.0     7.24    16.9
7.26    15.8     7.28    15.5     7.30    19.0     7.32    24.2     7.34    27.8
7.36    29.4     7.38    29.4     7.40    29.4     7.42    29.4     7.44    16.0
7.46    17.1     7.48    15.1     7.50    16.8     7.52    19.6     7.54    22.0
7.56    20.5     7.58    20.7     8.00    18.3     8.02    15.5     8.04    14.3
8.06    14.7     8.08    14.5     8.10    14.6     8.12    14.5     8.14    14.6
8.16    14.5     8.18    14.5     8.20    14.5     8.22    14.5     8.24    14.5
8.26    14.5     8.28    15.1     8.30    15.3     8.32    15.2     8.34    15.3
8.36    15.0     8.38    15.1     8.40    15.9     8.42    16.0     8.44    16.5
8.46    17.7     8.48    19.0     8.50    20.8     8.52    20.9     8.54    20.3
8.56    19.0     8.58    18.9     9.00    18.6     9.02    18.2     9.04    19.0
9.06    18.3     9.08    17.6     9.10    17.1     9.12    17.1     9.14    16.5
9.16    15.7     9.18    15.3     9.20    15.0     9.22     3.5     9.24     0.0


  E    R  LENGTH  SLOPE    N    DIA. CAPAC  VELOC  DESIGN INLET  RETENT  VOL.
           FEET     %            IN.  CFS    FT/S    CFS  Q-CFS   FT^3  SPEC.

  1    0    161   0.90  0.013  27     29.3   7.4     0.0  ----       0      0
                                                    33.6   33.6     685

 PONDING VOLUME AVAILABLE IN STREET (REACH) = 48162.114  FT^3

PONDING DEPTH
 0.72      INCHES IN STREET GUTTER
 0.00      INCHES IN ROADWAY (R OF W)
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MODELLING URBAN AND AGRICULTURAL DEVELOPMENT, HYDROLOGY AND EUTROPHICATION IN A LOUISIANA SWAMP FOREST ECOSYSTEM

C. S. Hopkinson*
J. W. Day Jr.

In the past 30 years there have been rapid changes in the structure and function of the ecosystems located in the headwater regions of Louisiana's coastal estuaries (Craig and Day 1977, Butler 1975, Gael and Hopkinson 1979). Freshwater lakes that were once clear and coffee-colored and that had abundant game fish populations (Lantz 1970) are now characterized as being green and odoriferous, having frequent algal blooms, occasional fish kills, and gizzard shad and catfish populations. Large portions of swamp and marsh have been reclaimed and marginally dry natural levee land drained for agricultural and urban use. An extensive canal network and channelized bayous lead to rapid storm water runoff from uplands to marsh lakes. This study investigates the relation between land use, hydrology, nutrient loading and biological productivity in the des Allemands swamp system.

DESCRIPTION OF AREA

The des Allemands swamp forest system is located in the headwaters of the Barataria Basin Hydrologic Unit. The roughly triangularly shaped unit is an interdistributary basin bordered by the Mississippi River, Bayou Lafourche, and the Gulf of Mexico (Figure 1). Natural levees form the perimeter of the basin and range in elevation from 9 m along the artificial levees of the Mississippi River to 0.5 m at the junction with the basin wetlands. Since the artificial leveeing of the Mississippi

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River overbank flooding no longer occurs, the only source of fresh water is rainfall, which averages 152 cm/yr. Water flow is from the natural levees into the interior wetlands and from the Mississippi River - Bayou Lafourche basin apex to the Gulf of Mexico. The Gulfward three-quarters of the basin is tidally influenced.

The des Allemands swamp forest system occupies the quarter of the Barataria basin most remote from the Gulf of Mexico (Figure 1). The embankment of U.S. Highway 90 forms a barrier between the swamp forest and the rest of the basin. The only significant connection is Bayou des Allemands. The swamp forest system is a freshwater system without any direct tidal effects. However, water levels are controlled to some extent by lower estuary conditions. For example, prolonged southeasterly winds raise water levels in the upper estuary.

An obvious feature of the area is the enormous number of linear canals (Figure 1), constructed primarily for agricultural drainage, petroleum activity (pipeline and rig access canals), and navigation. Drainage densities range from 2.9 in relatively natural areas of the swamp to more than 800 in sugarcane fields on the natural levee uplands (Gael and Hopkinson 1979).

The effects of the increased drainage network are manifested two ways, (1) increased eutrophication of water bodies (Gael and Hopkinson 1979, Seaton 1979, Day et al. 1977, Butler 1975, Lantz 1970, Craig and Day 1977), and (2) lowered swamp forest productivity (Conner and Day 1976, Conner unpublished). Gael and Hopkinson (1979) found a direct linear correlation between drainage density and trophic state condition of water bodies. As drainage density increased the trophic state also increased. The reason for this is that nutrient-laden runoff water from the uplands is rapidly discharged to swamp lakes without overland flow and cleansing through the swamp forest. The effects are also seen in decreased biomass and productivity and species changes of trees (Conner unpublished). Aerial infrared photographs of the region vividly portray this phenomenon. The reason is related to hydrologic changes. Canal construction requires the placement of spoil material that invariably ends up in the form of linear spoil banks running parallel to canals. This prevents overbank flooding, retards water and material exchange, and causes prolonged ponding and stagnation.

OBJECTIVES

In this section the following subjects are investigated: (1) urbanization and changing land use patterns in the des Allemands ecosystem in relation to storm water and nutrient runoff to the swamp, and (2) the effects of several hydrologic regimes in the wetlands on nutrient loading and eutrophication of Lac des Allemands. This is done using Stormwater Management Model (SWMM; EPA 1971). Specifically the following are considered: (1) present (1975) and future (1995) quantitative rates of nutrient and water runoff from the rapidly developing natural levee lands along the perimeter of the swamp as a result of changing land use; and (2) hydrologic changes from the present such as water levels and discharge rates that would result from (a) the removal of most spoil banks, and (b) the removal of all spoil banks and wiering of several major agricultural drainage canals; and (3) the problem of relating and synthesizing data suggestive of increased nutrient uptake in the swamp forest in a moving water overland flow regime. In particular this section deals with the question of by what what percentage nutrient loading to an already hypereutrophic Lac des Allemands can be reduced if swamp hydrology is modified.

THE SWMM MODEL

The SWMM is applied in this study to examine runoff and associated nitrogen and phosphorus nutrients from the natural levee uplands into the swamp and then to follow it through the swamp, through Lac des Allemands, and out of the system via Bayou des Allemands. Runoff from uplands is modelled with the RUNOFF block of SWMM and receiving effects in the swamp forest are modelled in the RECEIVE block (see Figure 2).

Nutrient accounting is an integral part of RUNOFF and RECEIVE blocks. In RUNOFF the availability of various nutrients is determined for the land surface. Nutrients are transported to the swamp in conjunction with water runoff from uplands. In contrast to RUNOFF, nutrients are treated nonconservatively in RECEIVE. Water flow advects nutrients from node to node, and depending on concentration and location are either added to or removed from the water column.

DES ALLEMANDS SYSTEM MODEL

LAND USE

Geometrical conceptualization of flow patterns and estimation of model coefficients and parameters are based on the areal extent of the various land use categories existing in the des Allemands system presently and in the future. The relationship of land use to hydrology is shown in the conceptual model (Figure 2).

Present and future areal extents of the major land uses in the des Allemands system are presented in Table 1. Of particular interest are the areas of various upland habitats. At present the majority of land on the natural levee is devoted to agriculture with forest land also being significant. Land devoted to urban activities such as residential, industrial, etc. comprises only 7 percent of the total. By 1995, however, based on growth projections of the Louisiana State Planning Office, land devoted to urban categories will increase by 9708 ha (321 percent). These projections vividly portray the imminent impact of the LOOP superport development. Present plans call for onshore storage facilities to be located in St. James Parish, which is within the study area. The majority of the increased urban area will be at the expense of agriculture land (70 percent). Smaller percentages will come from reclaimed swamp forest (20 percent) and upland forest (9 percent).

CONCEPTUALIZATION

Physical abstraction of the system is the first step in implementing the model (Figure 3). Separate procedures were followed for RUNOFF and RECEIVE. For RUNOFF, natural levee uplands were conceptualized into 11 different subcatchment areas, each based on all existing input locations of storm water runoff into the swamp. A subcatchment represents the drainage basin of an inlet to the swamp. Each subcatchment is unique in size, slope, and drainage density. Abstraction of the swamp, for RECEIVE, was more complicated. The swamp system was logically divided into three types of physical elements: lake, bayou or canal, and back swamp. In this finite element model each element is treated as a node(s) (or junction) that is (are) characterized by area, and elevation. The connection between nodes is represented as a channel that has a characteristic elevation, width, length, and roughness coefficient. The swamp is conceived as a few major channels, representing major agricultural drainage canals and channelized bayous, which lead to Lac des Allemands. The lake is represented by five nodes. Swamp area adjacent to these major channels is conceptualized as backswamp nodes that are separated from the channels by levees. Channels connecting backswamp to bayou or canal are represented in two ways. The major channel represents over-levee flow and has long characteristic widths and depths that are above the level of backswamp areas. Small breaks in levees are represented by channels with narrow widths and depths similar to backswamp elevations. This conceptualization reflects the effect of the enormous canal network: swamp impoundment. Currently most backswamps serve only as water storage areas. Water does not flow downstream at backswamp junctions. Rather water flows perpendicular to the main bayou. If several canals, levees or roads that run perpendicular to the downstream axis of the bayous were removed so that water could flow downstream from backswamp node to backswamp node. The conceptualization of this regime is shown in Figures 2 and 3.

PARAMETERIZATION PROCESS AND RESULTS

RUNOFF

Estimates were made of the coefficients and parameters that are necessary to characterize the hydraulic properties of a subcatchment such as surface area, drainage density, slope, infiltration rates (maximum, minimum, decay), detention depths, roughness coefficients, percent imperviousness, and canal morphology. Each subcatchment represents an idealized runoff area with uniform slope. It is assumed that each subcatchment has a unique area, drainage density, and slope. The other parameters are similar for each subcatchment and change only as land use does.

Hydraulic properties for the subcatchments were determined by first calculating them for each land use category and then by calculating an average based on the relative area of all land use categories on the subcatchments. Hydraulic properties assigned to each land use category are seen in Table 2. Parameters that vary the most for the different land uses are percent imperviousness of the land and the roughness coefficient for pervious land. Whereas about 100 percent of agricultural and forest lands are pervious only 30 percent of industrial lands are. Likewise pasture land has a high roughness factor while for urban and industrial areas it is lower. The overall picture that emerges is that urban land has hydraulic properties that will lead to rapid storm water runoff. Pasture land will have the least rapid runoff rate.

In Table 3 subcatchment parameters that represent an average of the various land use categories are listed. These were calculated using land use ratios calculated for 1975 and projected for 1995. Land use was not determined on a subcatchment basis but on a total upland basis. Therefore each subcatchment is treated identically. Most runoff parameters change little or none from 1975 to 1995. Minor differences were calculated for infiltration rates, detention depths on pervious areas, and roughness coefficients on pervious areas. The most significant hydraulic change will be the area of impervious land which increases from 4 to 13 percent.

Runoff from each subcatchment is collected in separate canals and then routed to the swamp. Output from the 11 canals serves as input to the RECEIVE block. Table 4 presents hydraulic information of each canal.

RECEIVE

Estimates for the RECEIVE block of swamp forest topography, junction areas, water depths, channel lengths and widths, and roughness coefficients were obtained by three methods. First, in the field the water slope from node 6 to 45 was measured using USGS benchmarks at nodes 18 and 6

(Figure 4) USACOE stage recorders at nodes 18 and 45. Surveyed levee heights along stream channels were referenced to MSL using the calculated water level of each node. Backswamp elevations and channel depths were calculated likewise. Backswamp slopes were calculated by surveying a transect from the uplands north of junction 20 to junction 20. It was determined that at the swamp-upland interface land slope is essentially 0. (2) Junction surface areas were calculated by digitizing USGS 15' quadrangle maps. Third, additional topographic and channel morphologic information was gathered from Louisiana Department of Public Works surveying and blue print drawings of more than a dozen major canal network systems that encompass most of the swamp forest.

To simulate water flow through occasional spoil bank breaks a parallel system of channels was simaltaneously used to connect two junctions. One channel is characterized by large negative depths (negative implies above MSL) and wide widths, while the other channel is characterized by small negative depths and small widths. These channels represent, in the former case, flow over the top of the spoil banks (possible only when water reached that elevation), and in the latter case, flow through bank breaks.

Water levels, areas, and bottom depths that describe each of the 45 junctions modelled are presented in Table 5. In Table 6 descriptive channel parameters are listed. Manning's roughness coefficients were estimated from SCS (1972). Backswamp channels had coefficients of 0.2, bayous 0.06, lake 0.014, and drainage canals 0.028. Channels changed for the simulation representing spoil bank and drainage canal removal are described in Table 7. Fewer parallel channels were used to abstract this condition and, therefore, there are only 73 channels versus 82 in the original setup. To represent the case of levee removal only, all parallel channels were dropped. Instead the wide channel widths associated with channels going over spoil banks were combined with depths of the spoil bank break channels.

NUTRIENT ACCOUNTING

Nutrient parameterization was quite involved. First, nutrient availability was determined by land use and then weighted with respect to total area to estimate total loading per year to the swamp. This was done using 1975 and 1995 land use areas. Loading was then divided into storm events and the coefficients relating to runoff determined. Last, uptake rates and equilibrium concentrations of N and P were determined for each junction in both stagnant and flowing backswamp hydrologic regimes.

Uplands

Nutrient availability on the upland subcatchments is a function of land use. The quantities of nitrogen and phosphorus expected to be available on various upland sources are listed in Table 8. With the exception of agricultural nutrient runoff all values are from the literature. Estimates from local studies, e.g. Craig and Day (1977) were used when available. Local estimates were available for P associated with sewage and urban runoff. In the former case the Louisiana value is considerably lower than that measured elsewhere (214 vs 795 g P/person/yr), while in the other the estimate is slightly higher. On an areal basis agricultural land contributes more N and P than any other land use. Forest and cleared lands contribute the least. The N to P ratio of these various sources varies from 155:1 for pastureland to 4.5:1 for cropland. Sewage, cropland, and urban lands are relatively rich with respect to potential P sources while pasture, cleared, and forest lands are richer with respect to N.

Agricultural nutrient runoff estimates were calculated by constructing N and P budgets for each major crop and pasture. Budgets were constructed assuming steady state conditions by solving material balances for unknown inputs or outputs. Considered sources and sinks were fertilizer, crop harvest, fire (volatilization), sediment erosion, and runoff. Runoff of nutrients was in all cases the unknown flow.

Figure 4 portrays nutrient budgets of cropland and pasture. N fertilizer inputs range from 0 to 209 $g/m^2/yr$ for soybeans and vegetable-orchard crops respectively. These same crops also had the lowest and highest P application rates. As sugarcane and pasture together represent more than two-thirds of total farmed agricultural land, it should be mentioned that both receive high loadings of N fertilizer. N and P that are removed from the land via harvest is highest for soybeans, a crop that receives no fertilizer. Nutrient loss is also high for sugarcane.

Nutrient loss associated with sediment erosion was assumed to be equal for all crops except pasture for which no sediment erosion was assumed. Nutrient loss was calculated by multiplying the weight of sediment lost with the percentage N and P of an average local soil type. The loss of sediment lost with the percentage N and P of an average local soil type. The loss ofsediment associated N and P was calculated to be 12 g N and 4.8 g P per m^2 per year. Sediment erosion rates were assumed to be the same as that measured for sugarcane land in the Lake Verret watershed (SCS 1976). Lake Verret is adjacent to Lake des Allemands and has similar soil types and agricultural practices. The SCS study reported that 2.55 kg sediment erodes from an average m^2 of agricultural land. Eighty-two percent of all sediment erosion in the watershed was attributed to agricultural land. Of this total 68 percent was deposited in field ditches. The remainder reaches the swamp. Of that, it was further calculated that 6.2 percent reaches the lake. Soil nutrient analyses of Golden and Ricaud (1965) for Mhoon, Commerce, and Sharkey soils which are found in the des Allemands area have N and P contents ranging from .055 percent to 0.175 percent for N and 243 to 362 ppm for P.

At this point solving the materials balance for nutrient content of each soil leads to substantial quantities of N and P in the soil which are unaccounted for. If soil fertility is assumed not to be increasing then the balance indicates that the surplus must be running off the fields either in dissolved or particulate form. This precludes denitrification. Patrick (personal communication) has done extensive agronomic work in Louisiana coastal soils and suggests, that to account for refractory N and P buildup in soils and denitrification, 10 percent of the applied N and 1 percent of the applied P be routed from cropland to the swamp. The range of values in Table 8 reflects what combination of estimates is used. I believe that the best estimate for cropland includes measured erosion plus 10 percent and 1 percent of the applied N and P fertilizer, respectively. For pastureland I use 10 percent of excess fertilizer plus Shannon and Brezonik's (1972) estimate for phosphorus.

In Table 9 calculated loadings to the swamp in 1975 and 1995 are shown. These figures were obtained by multiplying the expected loadings per unit area by the area of each land use category. Note that in 1975 about 75 percent of the N loading and 95 percent of the P loading comes from agricultural land. Urban- and sewage-derived nutrients are minor in comparison. In 1995 total loadings are projected to increase 28 percent for N and 16 percent for P. The most dramatic differences in 1995 will be with respect to N. Agricultural loading will decrease, while sewage related loading will increase. N loading related to sewage in 1995 is projected to be almost triple that of 1975. P loading in 1995 shows the same trend. P runoff from croplands will constitute only 61 percent of the total in 1995 versus 93 percent in 1975, while loading from sewage and urban areas will constitute 36 percent versus 14 percent of the 1975-1995 totals respectively.

These nutrient budget calculations represent only a first step in quantifying nutrient runoff to the swamp. Some sources and sinks, e.g. nitrogen fixation, rain associated nutrients, groundwater, and denitrification require elaborate sampling to be adequately evaluated. Estimates from the literature are unavailable for this area. Consequently, the budgets presented here are partial budgets; they estimate only gross supply.

I assume that the loading rate of the above calculated annual loadings is directly proportional to annual rainfall. If 60" of rain falls annually, then 1" will transport 1/60 of the annual nutrient loading budgeted. A value of 4.6 was used for the constant to determine how much and how fast a percentage of nutrients exit per unit runoff. This value dictates that 1/2 inch of runoff per hour will wash away 90 percent of the nutrients available per hour (EPA 1971).

Swamp

The fate of nutrients after reaching the swamp is a function of concentrations and locations. Nutrients are either released from the sediments to the overlying water or removed from the overlying water and sorbed to sediments or incorporated into biomass.

Nutrient uptake is considerably different in the swamp than it is in the aquatic system. For simplification it's assumed that uptake rates in the lake are similar to those in other aquatic areas such as canals and bayous. Both N and P are retained at 0.0014 per day in water bodies whereas only N is retained in the swamp (0.005 per day). Swamp uptake was calculated from a nutrient accounting model and measurements of surface water flux for the des Allemands Swamp (Kemp 1978). The N uptake rate he found at a "natural" location (.01/day) was decreased by an amount corresponding to the difference in surface water flux between that site and the impounded swamp (901/1851 m^3/month). The rationale operating is that nutrient retention is positively related to the development and extent of an aerobic surface layer at the soil-water interface. The rate of supply of oxygen to the interface is a function of turbulence or water movement (Howeler and Bouldin 1971). Hence with reduced water flux in the impounded area there is also a reduced aerobic zone.

Uptake in water bodies was taken as the average of estimates based on Butler's (1975) research in Lac des Allemands and empirically defined relationships of Kirchner and Dillon (1975). In a nutrient budget analysis of Lac des Allemands Butler found that 45 percent of incoming nutrients to the lake were removed from the water column. Kirchner and Dillon predict nutrient retention from a relationship between discharge and lake surface area (areal water load). For Lac des Allemands 55 percent of incoming nutrients would be retained annually.

The above information indicated that if the present impounded regime of the swamp forest system were replaced by one characterized by significant overland flow, nutrient uptake rates would increase in the swamp. A crawfish pond hydrologically managed to mimic overland flow was investigated by Kemp (1978), and his data has been used to estimate nutrient uptake rates in an overland flow regime. Water levels and constant sheet flow are controlled via pumps, levees, and drainage culverts. In April 1977 Kemp analyzed crawfish farm water for N and P along a transect from the input pump to an outlet culvert. Uptake rates for N and P were calculated by reconstructing the hydrologic regime. The owner provided information concerning farm area, water depth, drainage rates, and pumping rates (1.38 x 106 m2, 45 cm, entire volume/5 days, or 1.24 x 10^5 m^3/day, 1.42 x 10^5 m^3/day respectively). If the farm can drain at 1.24 x 10^5 m^3/day without maintenance of head differentials then with head maintenance drainage would be greater. It was assumed that the owner was pumping an average of 1.33 x 10^5 m^3/day. The average distance a parcel of water would have traveled from the pump to the culvert was measured from a USGS 7 1/2" map to be 1466 m. Velocity would then average 0.0034 m/s. Kemp's stations were a total of 1200 m apart. The concentration of P decreased 33 percent, and of N 34 percent as water moved those 1200 m. In the pond, the uptake rate of N and P per day was 0.083 and 0.080 respectfully.

Swamp sediments have a buffering effect on water column P. The amount of P in flood water and soils depends on the capacity of the soil to release P to a solution low in P and to remove it from a solution high in P (Patrick and Khalid, 1974). No direct research has been done in the swamp to date that indicates the equilibrium concentration where P is removed and added to the column at equal rates. However, the equilibrium concentration estimate was based on Seaton's (1979) extensive sampling in a backswamp location that receives little upland runoff.

The average lowest concentrations of N and P found in the flood water of this location were 0.41 and 0.07 mg/l respectively. As concentrations never go below this level and as allochthonous inputs are minimal, we can assume that at these levels nutrients are released to the water column. These concentrations are used in the model as minimal concentrations at all junctions. There was insufficient data available upon which to estimate what these concentrations would be in an overland flow regime with aerobic surface layers. Patrick and Khalid (1974) found that under anaerobic conditions more P is released from the soil into solution than under aerobic conditions. By assuming no difference I underestimate true nutrient uptake in an overland flow situation.

Initial concentrations are from Butler (1975), Kemp (1978), and Seaton (1979). Total-N and total-P concentrations were set at 1.54-0.25 mg/l for stream junctions, 1.24-0.32 mg/l for backswamp junctions, and 1.60-0.27 mg/l for lake junctions, respectively.

SIMULATION RESULTS

Simulations were made for validation and predictive purposes. Validation was done mainly by sensitivity analysis in RUNOFF and by comparing simulation results to field observations in RECEIVE. Twenty-four simulations of RUNOFF were made to assess the effect of (1) 1975-1995 land use changes on water and nutrient runoff and (2) differing initial soil water contents on runoff. Simulations were made of RECEIVE to analyze water stages and discharges under four hydrologic conditions: (1) present conditions using low initial water levels (stages below elevation of channel spoil banks), (2) present conditions using initial water levels 30 cm higher than 1 (above spoil banks), (3) conditions representative of spoil bank removal along Bayou Chevreuil, and (4) conditions representative of complete spoil bank removal and of upland discharge into backswamp areas rather than into drainage canals. Simulations were also made to compare storm water and nutrient runoff using 1975 and 1995 land use areas and to compare nutrient loading to Lac des Allemands using swamp hydrologic conditions as in 1 and 4 above.

VALIDATION

Runoff

To properly calibrate and verify a model with large data inputs such as RUNOFF, some amount of field measurements are necessary. Lacking extensive field measurements I can partially validate this model by showing that total runoff as calculated in the model for a single storm event falls within ±10 percent of the total runoff calculated using a static determination procedure (SCS 1972) that also relies on many estimates of area hydraulic properties. Additionally, by doing a sensitivity analysis those parameters that most affect the results can be identified. If changes in unmeasured parameters do not affect results too much, then one can have some degree of confidence in model predictions.

Parameters that are adjustable and need field evaluation are subcatchment width, area, slope, percent imperviousness, Manning's roughness, surface storage, and infiltration rates. Of the 11 parameters high confidence is had in 6: subcatchment width, area, slope, and infiltration rates.

Sensitivity analyses proved that with the exception of area and slope (which must be considered as accurate), the percent imperviousness and the minimum infiltration rate were the only variables that when raised or lowered tenfold had major effects on runoff. By contrast, resistance factors and surface storage depths did not change the total percent running off more than 1 percent for 2.8 cm rainfalls.

Changing subcatchment width an order of magnitude resulted in increasing runoff slightly less than a factor of two. Changing the maximum infiltration rate from an estimated maximum rate (signifying a dry soil) to the estimated minimum rate (signifying a saturated soil) resulted in a 1.9 fold increase in total runoff with a 2.8 cm rainfall. However, for a 3.8 cm rainfall, runoff increased only 1.1 fold. This implies that to accurately predict runoff from small rainfalls great precision is needed in both determination of the maximum rate for a particular soil and in determination of the present moisture content of a soil.

Lowering the minimum infiltration rate by three-quarters caused major changes in runoff volumes for 2.8 through 11.0 cm rainfalls. Although the estimated true value was measured in a similar adjacent system, future effort should be directed towards gaining accurate measurements from several locations in the des Allemands basin.

Of all parameters the percent imperviousness proved to be the most critical. However, as the primary goal of this study is to predict runoff changes associated with changing land use areas, comparative results will be valuable, as both are based on the same assumptions regarding the imperviousness of various land use categories.

As the parameters whose estimations I have the least confidence in are the least sensitive components of the model, simulation results can be considered to be reasonably accurate.

Receive

Calibration and verification of RECEIVE results proved difficult, as field measurements were few and crude. The following information was available against which model simulation results could be compared: (1) Monthly current measurements taken for a one year period at junctions 18, 37, and 42 (Butler 1975), (2) water levels in Bayou Chevreuil extrapolated from measurements at junctions 18 and 6 (this study), (3) one time measurements of velocity and discharge at channels 11-14 and 11-30 (this study), (4) yearly average discharge predicted by Light et al. (1973) in channels 18-16, 37-16, and 44-45, and (5) measurements of water level drop at junction 20 during a period in which the level was just below spoil bank crest (Whitehurst 1977).

RECEIVE is started from a standstill condition using measured or estimated water level values at each junction. The first day of simulation represents a transient state where water levels and velocities are approaching a dynamic steady state. The acceptance of hydraulic inputs from RUNOFF begins on the second day.

Model verification is made using present hydrologic conditions and low initial water levels, no inputs from RUNOFF, and results from day two. Butler (1975) recorded surface currents at junctions 18, 37, and 45 monthly for a one-year period. The range and average velocity at each location was as follows: 1-41 cm/sec - 22 cm/sec average; 5-30 cm/sec - 16 cm/sec average; 4-41 cm/sec - 20 cm/sec average, respectively. Current measurements taken at several depths did not consistently show logarithmic profiles with depth. Speeds predicted in the model were 13, 6, and 12 cm/sec at junctions 18, 37, and 45, respectively. Model results assume cross-sectional homogeneity in flow hence are expected to be less than surface measurements. Simulated speeds do agree favorably with the range observed in the field.

After two days of simulation water levels in Bayou Chevreuil (channel running from node 6 to 16) rose an average of 16 cm (SD = 8 cm) at each junction. Although water levels rose the overall water level slope from junction 6 to 18 fell only slightly (2.39 vs 2.46 $\mathrm{cm \cdot km^{-1}}$). These results suggest that initial water levels used in the model were somewhat in error, but that the parameterization of the hydraulic character of this portion of the swamp is reasonably accurate. Otherwise slopes would have changed more.

Several cross-sectional and vertical measurements of water currents were measured at the intersection of channels 11-14 and 11-30 to determine what percentage of flow followed each channel and to check model predictions. Average integrated velocities and discharges for the channels were 7.1 cm/sec - 6.4 m/sec and 10 cm/sec - 5.6 m3/sec, respectively. Model results show 8.2 cm/sec, 7.3 m^3/sec, and 12.2 cm/sec - 6.8 m^3/sec, respectively. The high degree of similarity among observed and predicted currents, discharges, and percentage of discharge taking each route lends confidence in the accuracy of the model. Roughly two-thirds of the flow goes down Bayou Chevreuil, while the remainder goes toward Bayou Boeuf.

Light et al. (1973) developed computer programs to evaluate the flow regime in the Lac des Allemands system. Mean annual discharge was plotted at various locations in the swamp. Results indicated that discharge into Lac des Allemands from Bayou Chevreuil (channel 19-16) averaged 11.0 m^3/sec. Discharge south from junction 45 was estimated at 27.0 m^3/sec. Although Light's results were not verified, agreement with present results should lend credence to both models. Results from day two of RECEIVE indicate the discharge from Bayou Chevreuil to be 9.6

m^3/sec. From junction 45 discharge was 7.1 m^3/sec. Agreement is excellent between discharge from Bayou Chevreuil. Slightly lower flow in RECEIVE is expected as low water conditions were simulated. Flow from junction 45 is controlled by the head differential between Lac des Allemands and lower Barataria estuary. A tidal function with zero amplitude is used in the model to represent the downstream head. The downstream head causes water to pile up in Lac des Allemands. Consequently, discharge is lower than that shown by Light.

Due to the complexity of factors controlling water levels at junction 45 (des Allemands) this analysis of hydrologic and nutrient RECEIVE simulation results is restricted to the Bayou Chevreuil basin. This includes backswamp and stream junctions upstream of channel 16-19. For this reason, developing an accurate representation of forcing functions acting at junction 45 can be avoided. The remainder of the des Allemands system can be used as a forcing function acting on Bayou Chevreuil.

Whitehurst (1977) measured water levels at junction 20 during April 1977. To see if RECEIVE accurately represents field observations, his measurements of water level decrease when initial water levels were just below streamside spoil bank crests were reviewed. Whitehurst's observations were separated into two components: (1) Evapotranspiration, and (2) water flow through spoil bank breaks. Evapotranspiration was calculated using Thornthwaite and Mather (1955) techniques and local newspaper (Morning Advocate, Baton Rouge, LA) listings of temperature. Total water level drop not due to evaporation was attributed to flow. In RECEIVE the same evaporation rate was used. Water levels decreased 1.1 times faster in RECEIVE than as measured by Whitehurst. This close agreement tends to verify model results. It is assumed that if there is close representation of junction 20 hydrology, other backswamp junctions will also be adequately modeled.

Agreement among simulation results of RECEIVE with available field data suggests that this system has been suitably abstracted and modelled. The verification process showed that flow from junction 45 is not necessarily accurate, and for that reason analysis of simulation results is restricted to the upper portion of the swamp system only: Bayou Chevreuil basin.

SIMULATIONS

Runoff

RUNOFF simulations were made to assess runoff and nutrient loading using three different rainfall rates (2.8, 5.6, 11.2 cm/storm) and five different infiltration rates under 1975 and projected 1995 land use makeups. These results are shown in Figures 6 and 7. The 2.8 cm rainfall represents actual hourly rates measured at Gonzales, LA, 1 March 1977.

Runs 1-15 represent present land use areas while 16-30 represent 1995 land use areas. Runoff increases with increasing rainfall using present and future land use areas. As the maximum infiltration rate is lowered from 4.1 cm/hr to 0.2 cm/hr total runoff increases. Runoff increases more with the 2.8 cm rain than with the 5.6 and 11.2 cm rains. Lowering the minimum infiltration rate from 0.2 cm/hr to 0.05 cm/hr leads to a much greater increase in runoff than a comparable decrease in the maximum infiltration rate.

The increase in runoff under a 1995 land use scenario is the most important result of these simulations. The 1975-1995 runoff difference is greatest with dry soil conditions and low rainfall. Runoff increases 4.2 times over what it was with 1975 conditions. With high rainfall, however, the increase is a more modest 7 percent over 1975 runoff volumes. There is less difference found between dry and wet antecedent moisture conditions using 1995 land use areas than when using 1975 areas.

As 85 percent of all rainfall episodes have 3.2 cm or less of rain per day the increase in impervious area by 1995 will have dramatic overall effects on runoff volumes to the swamp. Standing, stagnant, impounded backswamp areas are already experiencing productivity decreases. Additional runoff to the swamp can only exacerbate the problem.

Receive

Four RECEIVE simulations were made to assess storm water flow under different hydrologic regimes using RUNOFF as the driving force. Regime 1 (RUN 1) is based on a physical abstraction of present hydrologic conditions. Initial water levels are those measured in February 1978. Levels are low. Regime 2 (RUN 2) uses the same hydraulic properties as RUN 1 but starts with water levels 30 cm higher. In Run 1 water levels in the main streams are lower than spoil bank elevations. In Run 2 water levels are high enough to allow flow over many spoil banks. Regime 3 (RUN 3) describes conditions that simulate spoil bank removal in many places. Water flow directions and initial levels are the same as Run 1. Backswamp flow remains at right angles to major stream flow. Regime 4 (RUN 4) has been physically abstracted to represent runoff directly into backswamp areas. Major agricultural drainage canals are lacking, which allows possibility of downstream overland flow. All spoil banks are removed. Water levels begin similar to Run 1.

RUNOFF uses 1975 land use areas, a 5.6 cm rainfall, and a maximum infiltration rate of 0.81 cm/hr. This infiltration rate corresponds to the annual average moisture content of soils in the area.

Simulations 1 and 2 (RUN 1 - RUN 2) compare water levels and currents for ten days during and after a storm. Rainfall occurs during day 1. In Figure 7a water levels are plotted for three junctions along Bayou Chevreuil. Levels rise more at the upper portion of the basin (junction 4 and 11) than at the lower portion (junction 18). Bayou Chevreuil water levels rise more in RUN 1 than RUN 2. The reason for this is that in RUN 1 water levels are initially considerably below spoil bank crests. All water input from the uplands is confined to canals and streams. A large volume of water added to the small volume capacity of the streams causes levels to rise significantly. In RUN 2, however, stream levels are already above levee crests, and the large runoff volume is not confined to streams but is spread over the entire swamp. In both simulations the maximum water level rise occurs on the same day as the storm. An important difference between these runs is that most water levels recede faster in RUN 2 than RUN 1. This is because water that is impounded in the backswamp in RUN 1 is released slowly, while in RUN 2 water is released rapidly until levels approach that of spoil bank heights. This simulation shows that impoundment buffers stream water levels.

Figure 7b shows water levels at backswamp locations adjacent to the above stream junctions. Compared to water levels in the stream, less difference is seen between these runs. Water levels at junctions 12, 13, and 20 rise less in RUN 1 than the rise measured at adjacent stream junctions (Figure 7a). In RUN 2 the backswamp and the stream rise comparably. This shows that in RUN 1 water is confined for the most part to the streams while in RUN 2 water is rising everywhere. In both runs water levels at junction 13 reach a maximum at a day other than day 1. Stage recessions are more rapid in RUN 2 than RUN 1. This agrees with the previous observation concerning stream recession rates. The buffering effect of impoundment is shown. Slow release from the backswamp causes slow stage recession in streams. In RUN 2 discharge to Lac des Allemands is about 80 percent higher than RUN 1. Flow through backswamp channels, e.g., 18-20, 20-16, 18-21, 21-16 is much greater in RUN 2 than RUN 1. This is as expected as in RUN 1 water levels are not high enough to flow into and through the backswamp.

Two other simulations examine the effect of levee and/or upland drainage canal removal. RUN 3 has the same channel grid as RUN 1 but all parallel channels are omitted. Instead depths of channels into the backswamp are the same as the backswamp elevation itself. RUN 4 also simulates levee removal, but upland dishcarge to the swamp is via overland flow and not via canals. Canal removal along with associated levees allows water to flow parallel to original canal axes, thus allowing considerably more downstream overland flow in the backswamp. RUN 3 and 4 use RUN 1 initial water levels. These are two alternatives that land owners or the state could examine if an attempt to manage the swamp were deemed beneficial.

RUN 1, 3, and 4 hydrographs for Bayou Chevreuil junctions 4, 11, and 18 are shown in Figure 7c. Hydrographs for RUN 1 and 3 rise more in the upper basin than they do at junction 18. At junction 4 RUN 1 and 3 hydrographs rise more than two times higher than the RUN 4 hydrograph. At junction 11 all runs show about the same water level rise on day 1, but while they then fall in RUN 1 and 4 they continue to rise through day 10 in RUN 3. At the lower end of Bayou Chevreuil initial hydrographs are comparable, but then water levels continue to rise in RUN 4 while they fall in the other two runs.

Backswamp hydrographs for RUN 1, 3, and 4 exhibit a different pattern than that exhibited in the stream (Figure 7d). As mentioned previously backswamp water levels in RUN 1 do not rise as much as the adjacent bayou levels. In RUN 3 and 4, however, levels in the backswamp rise considerably. RUN 4 contrasts the pattern of RUN 1 the most. Water levels at junctions 12 and 13 rise more than they did in the bayou. This demonstrates the flux of water directly to the backswamp from the uplands. At node 13 in RUN 3 water levels rise for 9 days, as long as water levels rise in the adjacent bayou. At junction 20 hydrographs decline from day 1 maximums for RUN 1 and 3 and climb through day 9 for RUN 4. Hydrographs show that upper backswamp recession rates are greatest in RUN 4. Analysis of RUNS 1, 3, and 4 indicates that upstream water levels fluctuate most in RUN 4. In RUN 4 the upper Chevreuil basin drains rapidly and water piles up in the lower basin.

Discharge rates past junction 18 reveal several changes that occur when swamp geometry is changed. RUN 3 has 1.4 times the discharge of RUN 1 and RUN 4 is greater still. In RUN 4 70 percent of the total flow to junction 18 is via the bayou. The remainder flows through the backswamp as overland flow. By way of contrast in RUN 1 only ten percent of total flow to junction 18 is through the backswamp. Spoil banks effectively prevent overbank flooding and significant downstream, backswamp flow. Water drains rapidly from the upstream regions of Bayou Chevreuil in RUN 4 and piles up in the lower region. Heads at junction

18 are higher in RUN 4 than in RUN 3 or RUN 1. The higher head at 18 is partially responsible for maintaining greater discharge to the lake in RUN 4 than in the other simulations.

Nutrient Model

The effect of land use area changes on nutrient runoff from the des Allemands uplands was shown in Figure 6. With higher rainfall episodes runoff of N and P is close to that calculated statistically in Table 9. With rainfalls less than or equal to 2.8 cm/day, low soil moisture content leads to nutrient runoff that is up to 2.1 times greater under 1995 conditions than under 1975 conditions. Soil conditions for 1995 lead to increased water runoff and consequently causes increased nutrient runoff.

Nutrient loading to Lac des Allemands from the Chevreuil basin was determined using RUN 1 and RUN 4. The latter was run with projected 1995 land use areas and nutrient runoff estimates while the former uses 1975 conditions. RUN 4 was chosen because unlike RUN 3 it does experience considerable downstream overland flow. Many backswamp areas in RUN 3 experience no or very slow back and forth flow from the bayou. Backswamp current speeds averaged 0 in RUN 1, 0.49 in RUN 3, and 0.81 cm/sec in RUN 4. As nutrient uptake is proportional to the development of an aerobic surface layer, RUN 4 is most likely to lead to such development. Sparling (1966) showed that at velocities <0.4 cm/sec O_2 was depleted in a bog. At velocities <1.0 cm/sec waters remained agitated and saturated with O_2. RUN 4 has the highest backswamp velocities and comes closest to resembling conditions under which nutrient dynamics were determined in the crawfish pond.

The nutrient load to Lac des Allemands is calculated as the flux volume of water times concentration. Figure 8 shows total flux from day 1 through day 10 to Lac des Allemands from the Chevreuil basin. Not only is discharge greater in RUN 4 but it continues to increase through day 9. RUN 1 has peak discharge one day after the storm. In Figure 9 cumulative discharge for 10 days is plotted for RUN 1 and RUN 4. Total discharge is 22 percent higher in RUN 4 versus 1, and a larger percentage of floodwater flows through the backswamp than through the bayou. Although nutrient runoff into the swamp was projected to increase 25 percent and 18 percent respectively for N and P between 1975 and 1995, loading to the lake in 1995 under RUN 4 hydraulics is projected to actually decrease 23 percent for N and 28 percent for P. This readily shows the effect of increased uptake rates in backswamp areas. In RUN 1 23 percent of total flow passes along channel 16-21 while in RUN 4 28 percent does. Although more water flows through this channel in RUN 4 considerably fewer nutrients reach the lake. A larger portion of nutrients entering this backswamp junction are removed from the water column in RUN 4 than RUN 1. Consequently loading is lowered. The same occurs with P. Water flow increases in RUN 4 but because of enhanced nutrient uptake in an overland flow regime nutrient loading is lowered.

MANAGEMENT IMPLICATIONS AND CONCLUSIONS

The ramifications of RUN 4 results are immense. The estimated decrease in nutrient loading to Lac des Allemands warrants additional scientific research to further verify model results. Contrary to commonly held beliefs, simulation results suggest that removal of levees and agricultural drainage canals could actually increase the rate of water runoff to Lac des Allemands with no increase in flooding of agricultural fields.

Nutrient loading to Lac des Allemands is presently 3.0 gN/m2/yr and 4.0 gP/m^2/yr (Craig and Day 1977). RUN 4 results suggest that loading can be reduced to at least 2.3 gN/m^2/yr and 3.2 gP/m^2/yr if reduction rates modeled in the Chevreuil basin can be extrapolated to the remaining des Allemands system. As RUN 4 results are based on higher than present loading rates, the above reductions should be considered as conservative estimates. Shannon and Brezonik (1972) studied the relationships between lake trophic state and N and P loading rates. If loading can be reduced to RUN 4 levels then the lake trophic state could be reduced 43 percent. This kind of change would help return the lake to former conditions.

In addition to lowering the trophic state in Lac des Allemands, hydrologic modifications similar to those modelled in RUN 4 could also increase swamp forest productivity. In addition to fertilization the improved drainage regime would lead to enhanced forest growth. In the crawfish farm that has an artificially controlled hydrologic regime with drained periods, Conner (unpublished) found forest productivity to be twice that in uncontrolled and impounded areas of the swamp. As RUN 4 conditions could lead to conditions similar to those found on the crawfish farm one could expect similar productivity increases throughout the swamp.

The benefits realized by managing swamp hydrology are not restricted to the forest and lake but extend to the entire des Allemands system. Natural energy inputs associated with primary production could be increased approximately twofold. As primary production accounts for half of all natural energy inputs a doubling would increase total natural energy 1.3 times. With the level of industrial and urban growth predicted for the region this area will need the potentially available increase in natural energies to match the increase of purchased, human system energies. As the investment ratio between these two energies is already beyond the optimal value of 2.5 (Odum and Odum 1976, Hopkinson 1979) any increases in natural energies to match the higher quality purchased energies will enable more productive use of human system energies.

In summary this modelling exercise has shown that the projected urban and industrial expansion in the des Allemands system will lead to increased nutrient loading and increased storm water runoff to the wetlands. Increased runoff is attributed mainly to the increase in impervious areas. It will be most pronounced with the predominant rainfalls of under 2.5 cm. Modelling swamp hydrology showed that the present system of canals and spoil banks causes impoundment of swamp areas and does not optimize discharge rates to Lac des Allemands. Backswamp areas act as buffers, storing water for slow release. Simulations show that the hydrology could be managed to increase discharge rates to the lower estuary, to increase productivity of the swamp forest and to decrease lake eutrophication. This could be done by removing spoil banks and allowing upland runoff to pass through the backswamp in place of through drainage canals. Such management would be welcomed by nature lovers, sugar cane farmers, hunters, fishers, and those who appreciate the aesthetic appeal of healthy wetland ecosystems.

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Table 1. Present and projected land use in the des Allemands swamp basin.*

Land Use	1975	1995	Δ
	Area (hectares)		
Wetland Total	100,619	98,678	-1,941
Swamp	89,413	87,569	-1,844
Water	4,706	4,609	- 97
Lake	6,500	6,500	0
Dryland Total	61,700	63,641	+1,914
Residential, commercial, and services	4,162	12,694	+8,532
Industrial	228	1,404	+1,176
Cropland	20,698	17,459	-3,239
Pasture	14,135	11,930	-2,205
Field Support	9,086	7,639	-1,447
Forest	13,392	12,519	- 873

*Based on information in Hopkinson (1979), projection for 1995.

(From Louisiana State Planning Office)

Table 2. Estimated values of runoff parameters.

	LAND USE CATEGORY					
Parameter	Urban	Industrial	Crop	Pasture	Forest	Reference
Manning's resistance factor (n)						EPA (1971) Nix (1978) Chow (1964) SCS (1976)
Impervious land	0.014	0.014				
Pervious land	0.075		0.075	0.30	0.25	
Detention depth (cm)						EPA (1971) Nix (1978)
Impervious land	0.16	0.16				
Pervious land	0.38	0.38	0.38	0.48	0.38	
Horton's infiltration rate ($cm \cdot hr^{-1}$)						Nix (1978)
Pervious - Max*	3.81	3.81	3.81	5.08	3.81	
Min	0.18	0.18	0.18	0.25	0.18	
Decay rate	0.008	0.008	0.008	0.008	0.008	
Percent Impervious Land	50	70	0	0	0	EPA (1971)
Percent Pervious Land	50					La. State Planning Office

*Based on soil moisture content of zero.

Table 3. Average Runoff Parameters Used for Simulations Based on Percent Land-Use in Total Swamp Basin

Date	Subcatchment Number	Width (m)	Area (ha)	Percent Impervious	Slope	(n) Impervious	(n) Pervious	Detention Depth (cm) Impervious	Detention Depth (cm) Pervious	Infiltration Maximum	Infiltration Minimum	$(cm \cdot hr^{-1})$ Decay	$(\cdot sec^{-1})$
1975	1	10549	3434	4.26	0.0014	0.014	0.180	0.019	0.049	4.16	0.20	0.003	
1995		10549	3434	13.05	0.0014	0.014	0.162	0.019	0.048	4.06	0.19	0.003	
	2	18883	7456		0.001								
	3	4075	4015		0.0005								
	4	18386	6937		0.0012								
	5	6979	2561		0.0008								
	6	9931	3495		0.0007								
	7	11941	3141		0.0014								
	8	8476	1333		0.0014								
	9	13290	4745		0.0013								
	10	14726	4368		0.0010								
	11	14696	4971		0.0011								

TABLE 4. Hydraulic Properties of Canals Draining Des Allemands Uplands.

Canal Depth (m)	Tributary Subcatchment	Width (m)	Length (m)	Slope (m/m)	Manning Roughness	Overflow Depth (m)
1	1	18.3	2634	56915	0.028	2.4
2	2	18.3	4641	19418	0.028	2.4
3	3	14.6	1021	10044	0.028	2.1
4	4	18.3	4493	15000	0.028	2.6
5	5	14.6	1737	10044	0.028	2.6
6	6	14.6	2438	10004	0.028	2.7
7	7	12.2	2972	13057	0.028	2.4
8	8	12.2	2073	17410	0.028	2.4
9	9	9.8	3353	56915	0.028	3.7
10	10	9.8	3696	56915	0.028	4.9
11	11	24.4	3635	56915	0.028	4.9

Table 5 - SWMM Junction Parameters

Junction	Water level (m)	Area (ha)	Bottom depth (m)
1	1.21	5.91	0.36
2	1.01	10.7	1.21
3	1.16	5184.9	1.30
4	1.16	7500.0	1.29
5	0.82	19.6	-0.55
6	0.91	4.9	-2.44
7	0.72	15.7	-0.61
8	0.81	7193.3	0.56
9	0.73	13.2	-0.73
10	0.61	32.1	-2.68
11	0.91	19.9	-0.23
12	0.46	30.0	-1.22
13	0.61	3123.4	0.46
14	0.28	33.3	-1.46
15	0.56	1495.4	0.42
16	0.46	13.7	-1.1
17	0.33	1606.9	0.16
18	0.32	20.5	-1.52
19	0.37	31.3	-1.44
20	0.46	4818.6	0.33
21	0.46	3601.3	0.30
22	0.46	36.0	-0.91
23	0.32	6954.1	0.30
24	0.46	25.9	-0.91
25	0.32	3816.0	0.30
26	0.46	16.0	-0.91
27	0.32	6757.4	0.30
28	0.32	9084.6	0.30
29	0.32	13.4	-0.91
30	0.46	719.3	0.41
31	0.76	2303.8	0.36
32	0.37	2444.2	0.34
33	0.46	1211.9	0.58
34	0.32	4497.2	0.43
35	0.46	1815.1	0.58
36	0.32	3717.3	0.43
37	0.27	2958.2	0.27
38	0.23	1035.3	-1.52
39	0.23	1164.3	-1.52
40	0.23	785.3	-1.52
41	0.23	547.4	-1.52
42	0.23	2040.0	-1.52
43	0.23	489.4	-1.52
44	0.23	189.6	-1.52
45	0.23	98.4	-1.52

Table 6 - SWMM Channel Parameters

Channel	Coordinates*		Length (m)	Width (m)	Bottom depth (m)	Roughness Coefficient (n)
1	1	2	7544.8	15.2	0.358	0.06
2	2	5	2747.8	9052.6	1.164	0.2
3	2	7	2752.6	10638.0	1.164	0.2
4	2	3	6445.8	15.2	0.171	0.06
5	4	7	6836.6	7109.8	1.015	0.2
6	3	6	2967.2	32.0	-0.52	0.028
7	3	4	3337.0	36.3	-0.58	0.06
8	3	12	5172.2	4451.3	1.77	0.2
9	4	12	3632.3	3586.4	1.43	0.2
10	10	12	4362.3	4330.6	1.52	0.2
11	11	12	7626.7	4439.5	1.37	0.2
12	4	10	4305.3	30.3	-0.64	0.06
13	9	10	6527.0	28.3	-3.73	0.028
14	10	11	4556.2	42.7	-0.95	0.06
15	11	43	2516.4	9855.7	0.91	0.2
16	11	30	3126.6	30.5	-1.22	0.06
17	8	30	9648.1	18.3	-1.31	0.028
18	11	14	6283.1	45.7	-1.34	0.06
19	14	38	5376.7	21.3	-1.1	0.028
20	14	17	1813.3	2834.8	0.99	0.2
21	14	15	2165.6	7624.8	1.21	0.2
22	14	18	2757.8	45.7	-1.49	0.06
23	18	20	3459.2	4265.2	0.55	0.2
24	18	19	6076.8	45.7	-1.78	0.06
25	18	21	3678.0	4765.2	0.4	0.2
26	19	20	1710.8	6894.3	0.55	0.2
27	20	16	4302.7	6402.9	0.33	0.2
28	19	16	7139.9	45.7	-1.68	0.06
29	21	16	5491.3	2460.0	0.30	0.2
30	16	39	7805.6	1541.7	-1.52	0.014
31	22	39	8581.5	39.6	-0.91	0.028
32	22	23	8232.5	9.1	-0.46	0.06
33	23	39	2804.5	9081.8	0.29	0.2
34	39	40	6462.1	1250.0	-1.52	0.014
35	39	42	5021.4	3791.7	-1.52	0.014
36	40	42	5501.0	2983.5	-1.52	0.014
37	41	42	3409.8	3410.6	-1.52	0.014
38	42	43	4850.6	3311.6	-1.52	0.014
39	16	42	5291.6	4583.6	-1.52	0.014
40	26	43	6113.1	2125.1	-1.52	0.014
41	24	40	4186.6	30.5	-0.98	0.06
42	25	40	4596.6	6519.4	0.29	0.10
43	25	41	2585.6	7335.3	0.43	0.2
44	26	43	8450.9	21.3	-0.98	0.028
45	41	43	5106.3	1708.4	-1.52	0.014
46	27	43	5344.5	1343.3	0.4	0.2
47	30	32	8177.8	18.3	-1.22	0.06
48	31	32	7667.7	15.2	-1.37	0.06
49	33	34	4168.1	21.3	-1.46	0.028
50	34	37	7611.8	21.3	-1.95	0.028
51	32	37	10524.0	27.4	-1.83	0.06
52	35	36	4727.8	21.3	-1.46	0.06
53	36	37	6329.2	21.3	-1.95	0.06
54	37	16	6016.8	45.7	-1.37	0.06
55	28	45	3840.5	7821.2	0.305	0.2
56	27	28	2834.6	5943.6	0.305	0.2
57	27	43	6278.9	21.3	-0.99	0.028
58	43	44	4267.2	792.5	-1.52	0.22
59	44	45	3638.8	21.3	-1.52	0.22
60	19	21	1536.1	6509.0	0.43	0.2
61	9	38	7993.1	15.2	-1.07	0.028
62	3	5	6944.0	4943.2	1.0	0.2
63	40	41	4401.9	1666.6	-1.52	0.014
64	2	3	2747.8	1005.8	1.01	0.2
65	2	4	2752.6	2709.4	1.01	0.2
66	5	8	5172.2	137.5	0.85	0.2
67	7	8	3632.3	110.9	0.7	0.2
68	8	10	4362.3	133.8	0.58	0.2
69	8	12	7626.7	138.1	0.46	0.2
70	12	13	2516.4	1095.2	0.48	0.2
71	14	17	1813.3	315.2	0.38	0.2
72	14	15	2165.6	869.3	0.41	0.2
73	19	20	1985.2	766.3	0.305	0.2
74	30	32	6603.8	4063.9	0.37	0.2
75	31	32	6605.6	3048.0	0.46	0.2
76	33	34	3627.7	6142.9	0.49	0.2
77	36	37	7025.3	6812.8	0.37	0.2
78	32	37	9520.4	2275.6	0.325	0.2
79	35	36	4572.0	8523.7	0.49	0.2
80	36	37	5120.6	5446.8	0.37	0.2
81	37	16	5503.5	4111.8	0.27	0.2
82	19	21	1536.1	723.3	0.30	0.2

* - refers to upstream - downstream terminus

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Table 7 . SWMM Channel Changes for 1995 Simulation.

Channel	Coordinates*		Length (m)	Width (m)	Bottom depth	Roughness Coefficient (n)
2	2	5	2747.8	9052.6	1.01	0.2
3	2	7	2752.6	10838.0	1.01	0.2
6	3	6	2967.2	12192.0	0.88	0.2
8	3	12	5172.2	4451.3	0.85	0.2
9	4	12	3632.3	3584.4	0.70	0.2
10	10	12	4362.3	4330.6	0.58	0.2
11	11	12	7626.7	4459.5	0.46	0.2
13	9	13	8158.6	5275.2	0.48	0.2
15	11	13	2516.4	3285.1	0.48	0.2
17	8	30	9648.1	18.3	0.30	0.2
19	13	38	3172.7	3996.5	0.48	0.06
20	14	17	1813.3	1378.9	0.38	0.2
21	14	15	2165.6	2608.2	0.41	0.2
23	18	20	3459.2	4765.2	0.30	0.2
25	18	21	3878.0	4765.2	0.30	0.2
26	19	20	1710.8	6894.3	0.30	0.2
31	24	23	1090.3	12965.6	0.30	0.2
41	24	40	4166.6	30.5	0.76	0.2
44	8	12	3925.5	6978.7	0.58	0.06
60	19	21	1534.1	7232.3	0.30	0.014
62	3	5	6944.0	4943.2	1.01	0.2
64	26	27	6019.2	9375.0	0.30	0.2
65	25	26	4012.7	5320.6	0.30	0.2
66	10	13	2642.0	3285.1	0.58	0.2
67	13	14	5032.9	3285.1	0.41	0.2
68	11	15	3176.9	2608.2	0.48	0.2
69	15	18	3421.4	2608.2	0.34	0.2
70	13	17	7209.4	1619.1	0.44	0.2
71	17	18	3299.2	1378.9	0.34	0.2
72	22	23	7484.4	1955.0	0.40	0.2
73	24	40	4166.6	2072.6	0.40	0.2

* - refers to upstream - downstream terminus

Table 8. Expected quantities of nitrogen and phosphorus from various sources.

Loading Source	Nutrient				Reference
	N	Ratio	P		
Sewage	3940 g·person^{-1}·yr^{-1}	5:1	795 g·person^{-1}·yr^{-1}		Shannon and Brezonik (1972)
			✓214 g·person^{-1}·yr^{-1}		Craig and Day (1975)
Cropland	✓2.37	4.47:1	✓0.53 g·m^{-2}·yr^{-1}*a		Hopkinson and Day (1979)
	2.26	124:1	0.018		Shannon and Brezonik (1972)
	1.2	2.3:1	0.5	*b	Hopkinson (1979)
	4.9	2.9:1	1.7	*c	Hopkinson (1979)
	6.1	2.8:1	2.2	*d	Hopkinson (1979)
Pasture	✓2.8	155:1	✓0.018	*e	Hopkinson (1979)
	0.85	47.2:1	0.018		Shannon and Brezonik (1972)
	10.9	--	0	*f	Hopkinson (1979)
Cleared	✓0.18	30:1	✓0.006		Shannon and Brezonik (1972)
Forest	✓0.24	40:1	✓0.008		Shannon and Brezonik (1972)
Urban	✓0.88	8:1	0.11		Shannon and Brezonik (1972)
			✓0.15		Craig and Day (1975)

*a = includes 10% and 1% of applied N and P respectively and measured erosion.

*b = erosion only.

*c = fertilizer excess only.

*d = erosion plus fertilizer excess.

*e = 1% of applied N, and Shannon and Brezonik (1972) estimate for P.

*f = fertilizer excess only.

✓ = values used in model.

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Table 9. Nutrient loading to swamp forest, and percentage contributions from various cultural and natural sources for the des Allemands Basin. Values in gN and P x 10^6.

		LAND USE CATEGORY						
Year	Nutrient	Forest	Cleared	Pasture	Crop	Urban	Sewage	Total
1975	N Load	32	76	396	491	38.6	222	1,200
	percentage	2.6	1.3	33.4	40.9	3.2	18.6	100
	P Load	1.07	0.55	2.54	110	6.59	12	130
	percentage	0.8	0.4	1.9	82.8	5.0	9.1	100
1995	N Load	30	13.8	334	414	124	615	1,530
	percentage	2.0	0.9	21.8	27.1	8.1	40.1	100
	P Load	1.0	0.46	2.15	92.5	21.1	33.4	151
	percentage	0.7	0.3	1.4	71.3	14.0	22.3	100

Figure Legends

Figure 1. Map of the des Allemands system showing its position in the Barataria Basin, the natural levees or uplands, the interior wetlands, and the enormous canal network.

Figure 2. Conceptual model relating the influence of land use characteristics on storm water and nutrient runoff from upland areas to the receiving wetlands.

Figure 3. Physical abstraction of the des Allemands hydrologic regime. Uplands are portrayed as 11 subcatchments that drain into interior wetlands. Wetlands are portrayed as either lake, stream or backswamp elements. Channels with x's across them were deleted in simulations representing overland flow.

Figure 4. Mass balance analyses of agricultural crops.

Figure 5. Effects of several factors on storm water runoff from uplands. Simulations are aggregated in three groups: (1) 1-5 and 16-20, (2) 6-10 and 21-25, (3) 11-15 and 26-30. Rainfall increased from group 1 to 3. Numbers less than 16 are based on 1975 land use, while those greater than 15 are based on 1975 land use. In each group the left most figure represents maximum and minimum infiltration rates of 4.1 and 0.5 cm/hr. The next 2.0 and 0.5, then 1.0 and 0.5, then 0.5 and 0.5, and finally 4.1 and 0.13 cm/hr, respectively. The first four represent a sequence of increasing initial soil moisture content.

Figure 6. Projected relative changes in N and P runoff loads to the des Allemands swamp from 1975 to 1995. See legend of Figure 5 for explanation.

Figure 7. Hydrographs at selected stream and backswamp areas. See text for explanation of four different runs.

Figure 8. Comparative daily discharge rates from the Bayou Chevreuil drainage basin to Lac des Allemands in RECEIVE RUNS 1 and 2.

Figure 9. Comparison of storm water and nutrient inputs to Lac des Allemands following a single storm event using 1975 and 1995 conditions.

Figure 1.

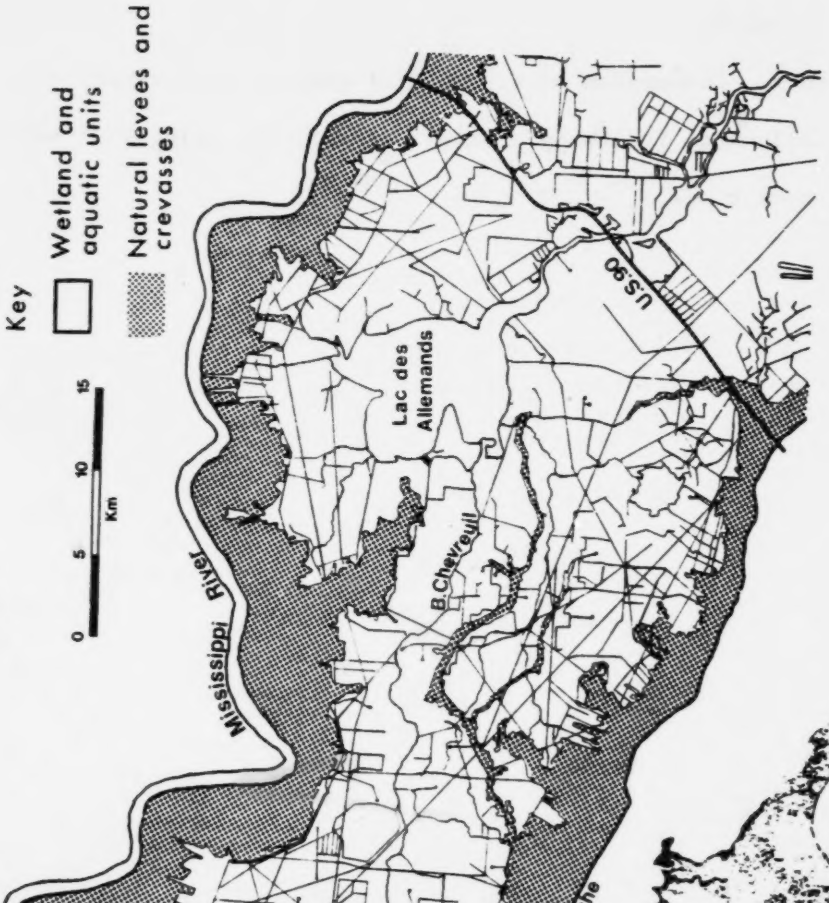

0
5
10
15
Km
Key
Wetland and aquatic units
Natural levees and crevasses
Mississippi River
Lac des Allemands
B.Chevreuil
U.S. 90
Bayou Lafourche
GULF OF MEXICO
N

Figure 2.

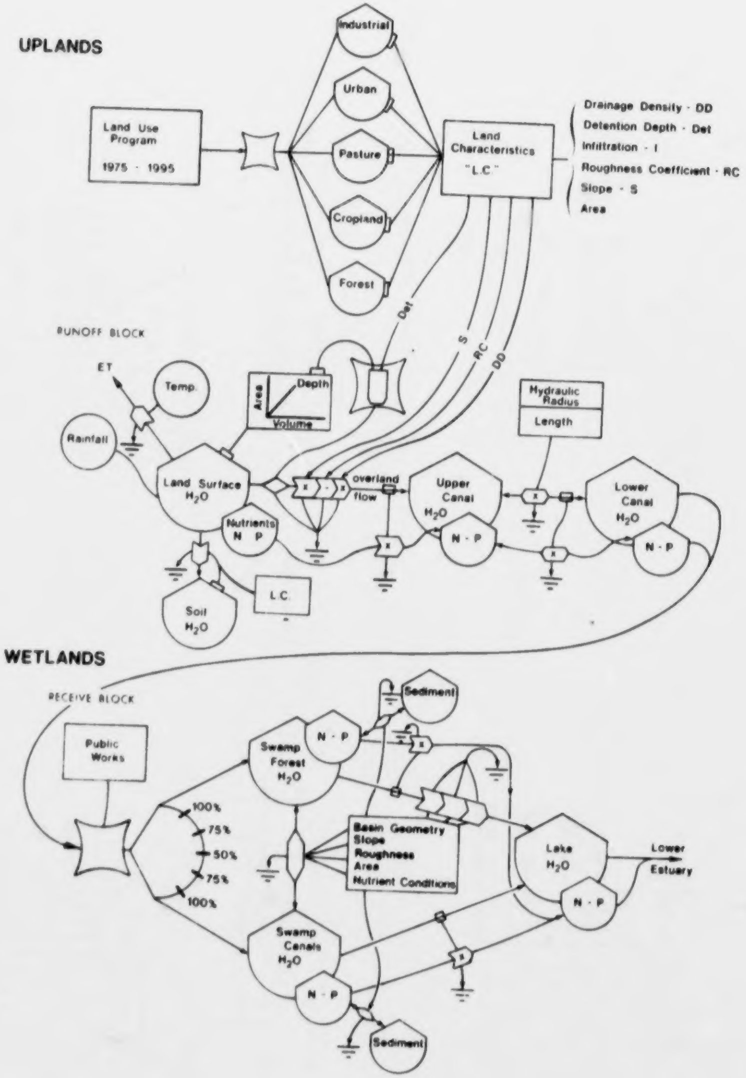

UPLANDS
Land Use Program
1975 - 1995
Industrial
Urban
Pasture
Cropland
Forest
Land Characteristics
"L.C."
Drainage Density - DD
Detention Depth - Det
Infiltration - I
Roughness Coefficient - RC
Slope - S
Area
RUNOFF BLOCK
ET
Temp.
Rainfall
Area
Depth
Volume
Det
S
RC
DD
Land Surface H_2O
Nutrients N P
overland flow
Upper Canal H_2O
N - P
Hydraulic Radius
Length
Lower Canal H_2O
N - P
Soil H_2O
L.C.
WETLANDS
RECEIVE BLOCK
Public Works
100%
75%
50%
75%
100%
Swamp Forest H_2O
N - P
Sediment
Basin Geometry
Slope
Roughness
Area
Nutrient Conditions
Lake H_2O
N - P
Lower Estuary
Swamp Canals H_2O
N - P
Sediment

Figure 3. Physical Abstraction of Lac Des Allemands Hydrologic Regime

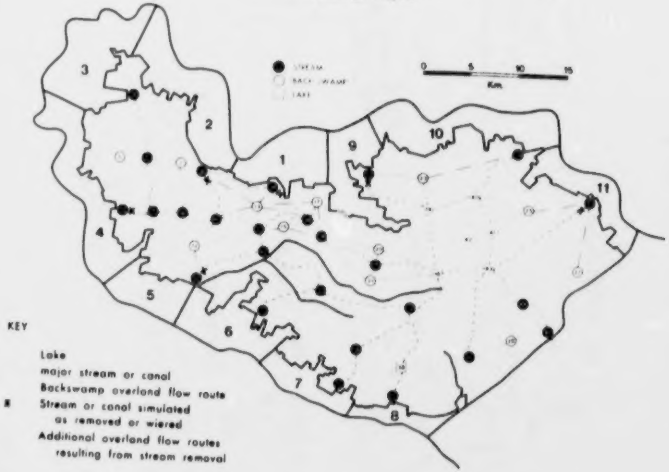

0
5
10
15
Km.
STREAM
BACKSWAMP
LAKE
1
2
3
4
5
6
7
8
9
10
11
KEY
Lake
major stream or canal
Backswamp overland flow route
x Stream or canal simulated as removed or wiered
Additional overland flow routes resulting from stream removal

Figure 4.

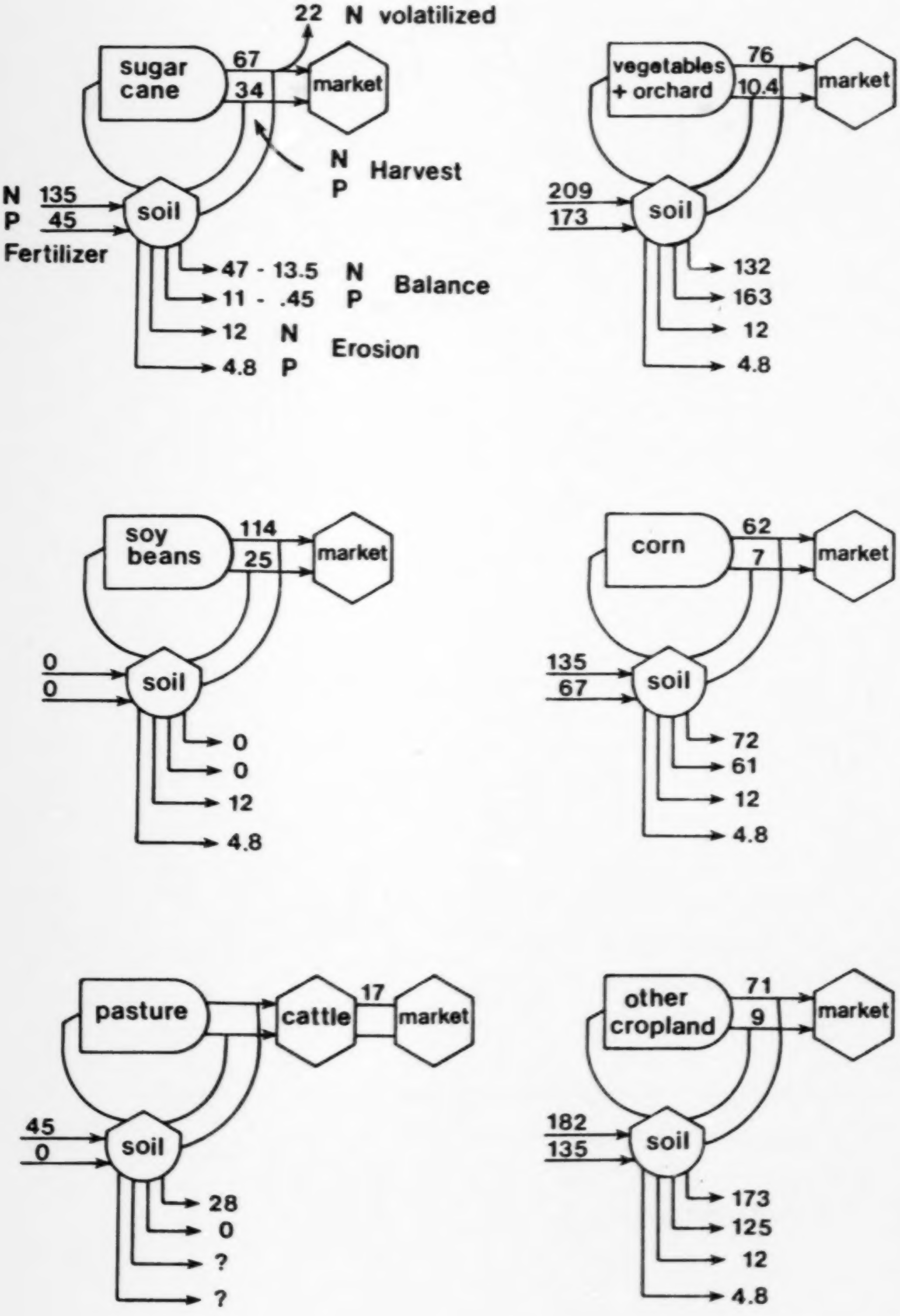

22 N volatilized
sugar cane
67
34
market
N
P
Harvest
N 135
P 45
Fertilizer
soil
47 - 13.5 N
11 - .45 P
Balance
12 N
4.8 P
Erosion
vegetables + orchard
76
10.4
market
209
173
soil
132
163
12
4.8
soy beans
114
25
market
0
0
soil
0
0
12
4.8
corn
62
7
market
135
67
soil
72
61
12
4.8
pasture
cattle
17
market
45
0
soil
28
0
?
?
other cropland
71
9
market
182
135
soil
173
125
12
4.8

Figure 5. RUNOFF COMPARISONS

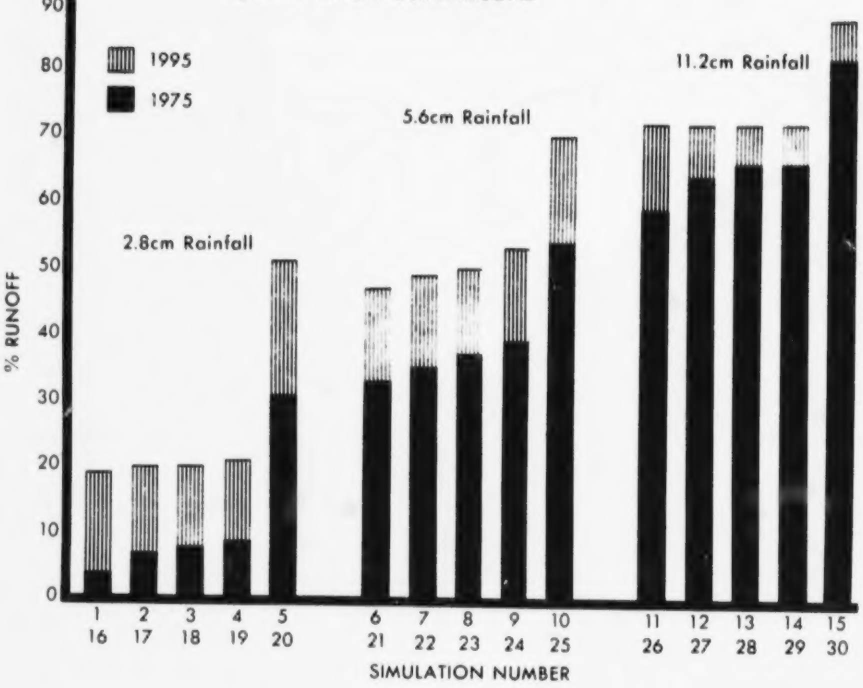
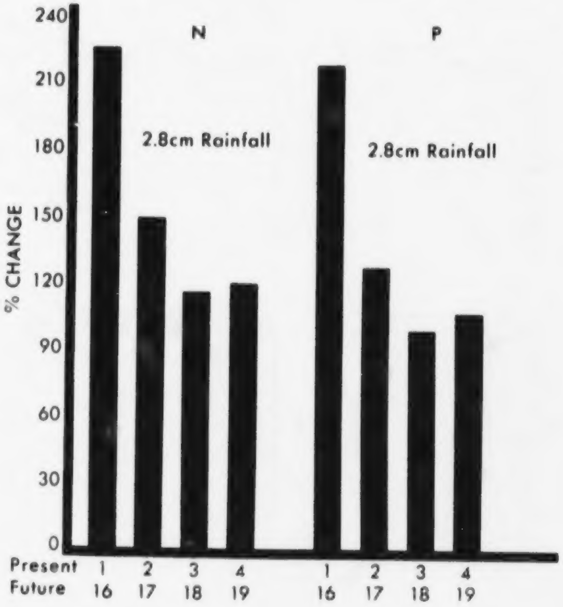
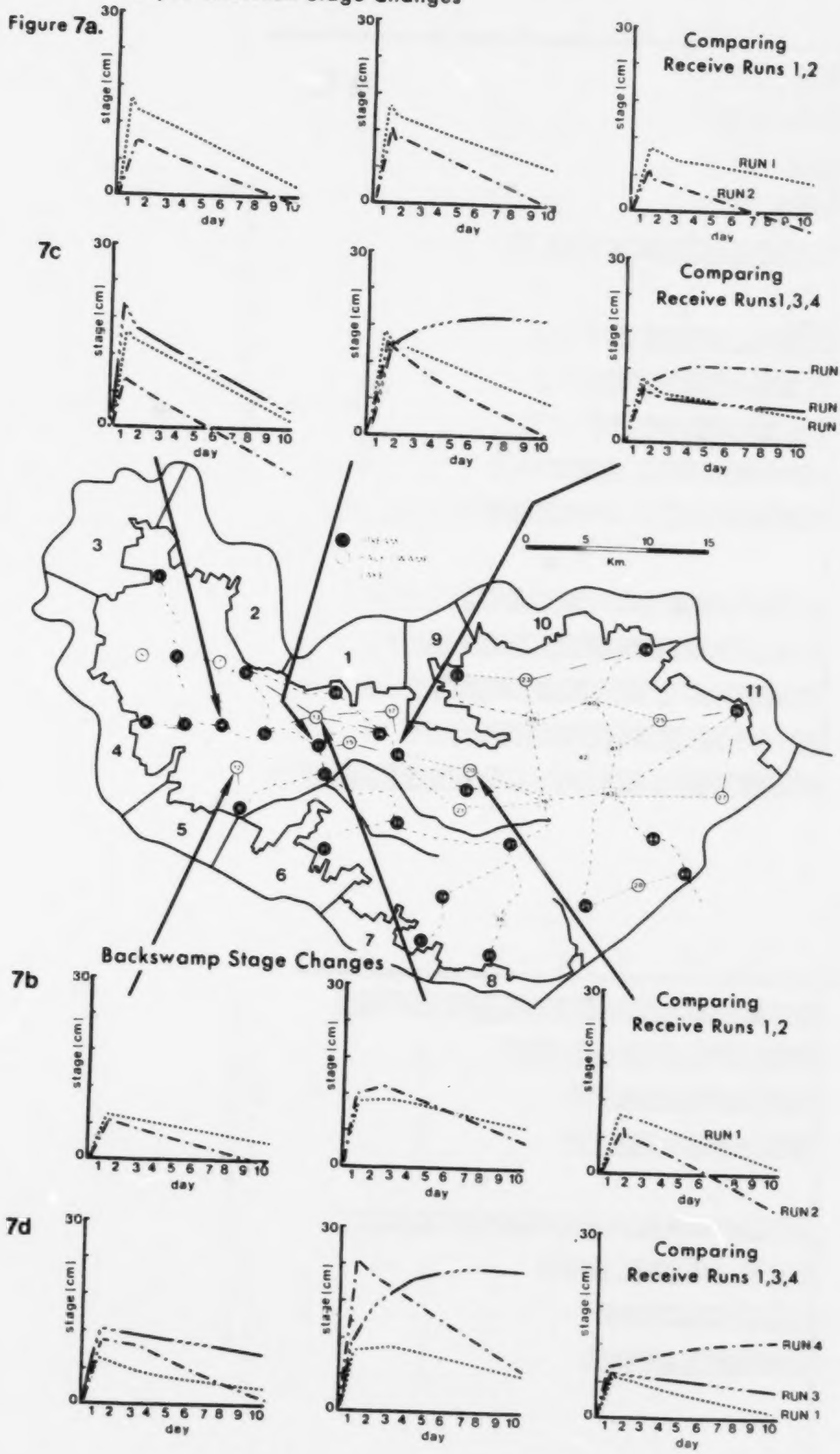


Figure 6. PROJECTED RELATIVE CHANGES IN FUTURE N AND P LOADINGS (1975-1995)




Bayou Chevreuil Stage Changes
Figure 7a.
7c
Backswamp Stage Changes
7b
7d
Comparing
Receive Runs 1,2
Comparing
Receive Runs1,3,4
Comparing
Receive Runs 1,2
Comparing
Receive Runs 1,3,4
stage (cm)
day
0
30
1 2 3 4 5 6 7 8 9 10
RUN 1
RUN 2
RUN 3
RUN 4
0 5 10 15
Km
1
2
3
4
5
6
7
8
9
10
11

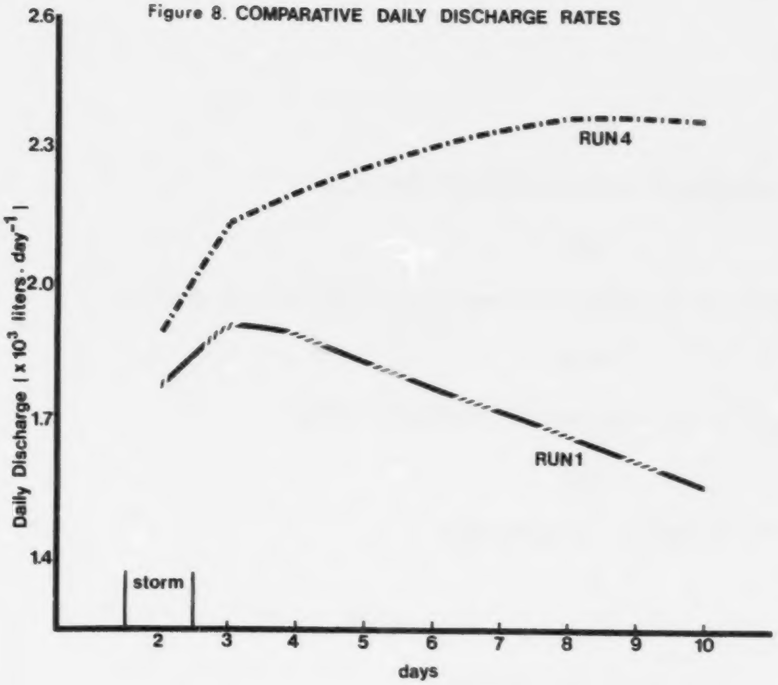
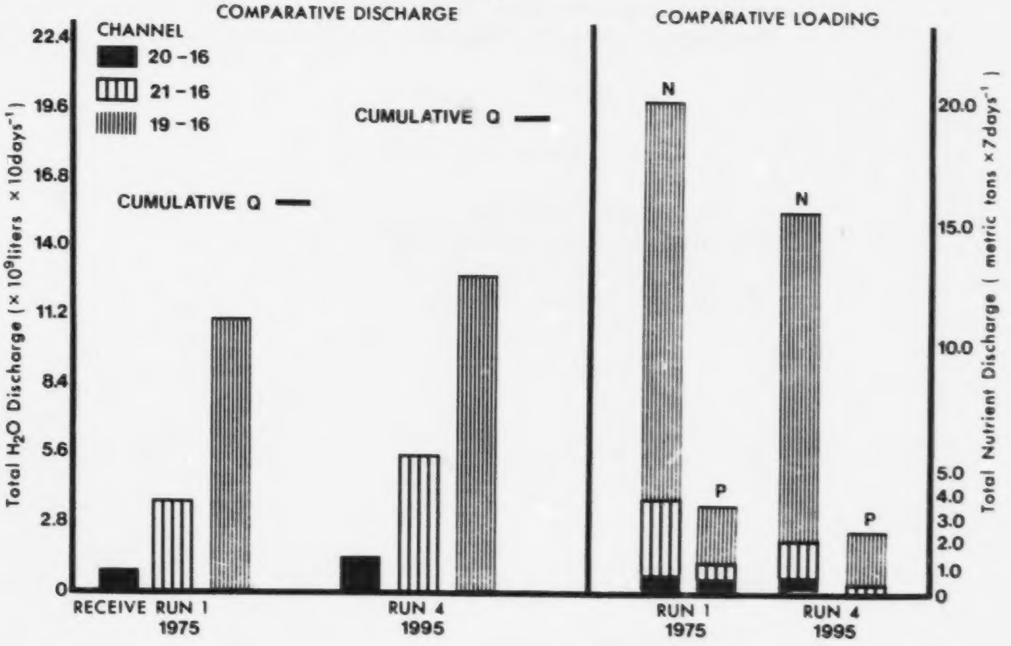


Figure 8. COMPARATIVE DAILY DISCHARGE RATES

Figure 9.



DEVELOPMENT AND IMPLEMENTATION
OF
AN URBAN-RURAL SUBCATCHMENT HYDROLOGIC MODEL (SUBHYD)
FOR
DISCRETE AND CONTINUOUS SIMULATIONS
ON
A MICRO COMPUTER

BY: Larry R. Thompson

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SWMM Users Group Meeting

Montreal, Quebec

May 24-25, 1979

INTRODUCTION

Hydrologic scientists have sought to understand and reduce the hydrologic cycle to a series of quantified physical laws in order to better predict the response of a watershed to rainfall. To this end a great deal of basic research has been done to identify and quantify various aspects and components of the hydrologic cycle in order that hydrologic modellers might be able to simulate watershed response to rainfall.

Unfortunately, the complexity of the processes involved have so far largely eluded exact quantification. However, reasonably sophisticated conceptual and semi-empirical methods have been developed to mathematically describe virtually every element of the hydrologic cycle which is of interest to the practising hydrologist.

Numerous of these techniques of modelling various components of the hydrologic cycle have been available for decades, but in the past it was not practical to estimate watershed response by means of linking mathematical descriptions of each hydrologic component. This was so because the shear magnitude of the calculations involved was far too great for manual applications. The advent and widespread use of computers is now revolutionizing applied hydrology by making it more practical to estimate watershed response simply by representing and linking each of the components of the hydrologic cycle mathematically.

The availability of computers has led to the development of a great number of hydrologic computer models in recent years. These have ranged in complexity from those that were simply computerizations of previous manual techniques such as the U.S. Department of Agriculture Soil Conservation Service (S.C.S.) curve number method (1954)[10] to those such as the U.S. Environmental Protection Agency's (E.P.A.) Storm Water Management Model[2] that attempted to simulate what were felt to be the most important aspects of the hydrologic cycle. This proliferation of models has been further added to by the fact that there are many special purpose models and also numerous versions of many of the models, in use.

Unfortunately as a result of this boom in model development there are no longer standard methods in hydrology which are consistently applied to various situations. Not infrequently two hydrologists have obtained somewhat different results when applying the same model to the same simulation, a fact which has tended to discredit hydrologic modelling in the eyes of various governmental officials that do not have a thorough understanding of the models themselves.[21] Model builders themselves have been partly to blame for this lack of consistency by not selecting parameters carefully enough so that users will fairly readily agree on the value of a given parameter.

Calibration of hydrologic models in order to better match a given observed runoff response has been a fairly common practice. Even very sophisticated distributed models such as that developed by Gupta and Solomon[5] have placed reliance on calibration in general and sometimes calibration parameters in particular in order to simulate watershed response to rainfall. Unfortunately the process of calibration has numerous pitfalls, particularly for models which assume constant travel time. Although many hydrologists realize the shortcomings of calibration, users of hydrologic modelling services are very often not aware of the following resulting pitfalls.

- Misleading ideas regarding the accuracy of a given model in a given situation are obtained.

- The general applicability of the model may be endangered. The model may become valid only for certain geographic locations and conditions.

- The model is apt to be calibrated to runoff events of fairly frequent magnitude but then used to determine the effects of a very large storm that may occur only once in a hundred years. If a model has had to be calibrated to the relatively small storms for which data exists there is virtually no guarantee that it will be reliable for extreme runoff events. This factor is of particular importance to models based upon unit hydrographs or runoff isochrones if these do not allow for the fact that speed of runoff and therefore peak flow increases with rainfall intensity.

If after checking that input data values are reasonable, a model requires calibration, it is in general due to one of the following factors:

- a fundamental runoff mechanism has been incorrectly assumed to be unimportant or has been inadequately simulated. Calibration in this case may allow the modeller to produce more reliable estimates of peak flow but will not improve knowledge of how runoff arrived at the outfall, a question of vital importance if water quality modelling is ever to be attempted.

  Rather than calibrate in such a case, the model builder should attempt to identify those underlying mechanisms which were neglected and incorporate a representation of them if a true increase in model accuracy is to result.

- meteorological or flow data was inaccurate or did not account for spatial variation over the watershed. Calibration in such a case will serve no useful purpose for it is highly unlikely that the same spatial distribution of rainfall or combination of measurement inaccuracies will occur consistently in the future.

- the model may have certain calibration parameters intended to tailor mathematical description of processes such as infiltration to a particular watershed. Once calibrated such a model is only truly valid for that watershed. This factor limits the strict applicability of such models to watersheds having a network of stream and rainfall gauges.

A further problem in hydrology which has become more pronounced since the advent of computer simulation models has been the artificial separation of the study into urban hydrology and rural hydrology. This has been largely due to the fact that various governmental agencies funding hydrologic model research and development have specific areas of responsibility. For example, when the United States Department of Agriculture (USDA) has supported the development of hydrologic models, the resulting models have been primarily applicable to agricultural watersheds. Also since urban hydrologic modellers are primarily concerned with effects of urbanization they have sometimes expressed the viewpoint that urban simulation models need not be valid for rural situations.

There has thus been a tendency on the part of hydrologic model builders to develop a rural model or an urban model, rather than just a hydrologic model. However, even to model the effects of urbanization necessitates the prediction of watershed response when the watershed was in a rural state before urbanization.

There has been a difference however, in the modelling techniques which have proven successful in both urban and rural hydrology. Successful urban hydrologic modelling has been characterized by careful conceptual simulation of impervious runoff, the dominant urban runoff mechanism. On the other hand, due to the complexity of the problem and multiplicity of relevant runoff mechanisms rural hydrologic modelling has tended to place greater reliance on traditional techniques such as unit hydrographs, isochronal methods, and hydrograph recession constants.

Differences between urban and rural hydrology and even between hydrology in diverse geographical locations are not however, due to any true fundamental differences, but rather to the varying relative importance of various runoff mechanisms and components in the hydrologic cycle.

The component in hydrologic modelling which has the greatest need for improvement is that which quantifies the runoff from an incremental area or subcatchment. It is generally felt that there are numerous good routing techniques available and that in most cases the errors introduced in flood routing calculations are minor compared to those in subcatchment hydrologic modelling.[9] A hydrologic model which consistently predicted watershed response within 20% has been considered good.[13,21]

Based upon factors brought out in the preceeding discussion it was felt that work in hydrologic modelling would be most profitably spent in the development of a better subcatchment hydrologic model which could become a component in any of the various comprehensive models currently available.

DESIGN CRITERIA FOR A SUBCATCHMENT HYDROLOGIC MODEL

After general reading and discussions with hydrologists and municipal engineers in the academic, consulting and government communities the following list of criteria for a subcatchment hydrologic model were drawn up.

1. The subcatchment model should be applicable to both urban and rural areas.
2. It should be a conceptual simulation model taking advantage of modern computer capabilities and not just a computerization of an existing manual technique.
3. The model should be able to simulate runoff from subcatchments ranging in size from a section of street to at least 20 sq. km.
4. The data required should be readily available from existing data sources. Parameters should be chosen and methods for their determination developed in such a way that various users will readily agree on the value of any given parameter. The data sources which are readily available to the hydrologist in Ontario are:

    - topographic maps
    - county soil maps
    - official plans and land use maps
    - aerial photographs
    - existing tabulations relating hydrologic parameters to one of the above sources
    - rain gauge, stream gauge, pan evaporation and temperature data

    Any soil parameters which are chosen should be readily measurable in the field should a user so desire.
5. The hydrologic cycle should be well enough represented that the model will give good results while using reasonable values for all parameters. (i.e. if accurate values of all variables are input, reliable results should be obtained thout calibration.)

6. The model should not assume that the speed of runoff is constant. Allowance must be made for the fact that runoff speed increases with intensity of rainfall and also for the effect of this factor on peak flows.
7. The model should be capable of continuous simulation and not restricted to discrete storm events.
8. In order to facilitate the acceptance of the model by the hydrologic modelling community, accepted methods of representing each of the runoff mechanisms and components of the hydrologic cycle should be used wherever possible.
9. While meeting the preceeding criteria the model must not use so much computer time and resources that it becomes too expensive for practical use.

IMPLICATIONS OF VARIOUS OF THE DESIGN CRITERIA

Criterion number 4 concerning the availability of data almost immediately disqualified the use of highly rigorous finite difference schemes to evaluate subsurface flow processes by solving the equations of flow in saturated and unsaturated porous media. There is simply not sufficient high quality data or inexpensive large computer resources available to the practicing hydrologist to justify the use of such techniques at this time. The approach has been taken throughout the model development that the sophistication to be employed on a given mechanism should be correlated to the quality of the associated portion of the data base.

Criterion number 6 concerning the avoidance of methods which assume constant travel time precludes the use of unit hydrograph and isochronal methods of overland flow calculation for these methods are based upon assumptions of constant travel time.

Criterion number 9 at first glance appears to be in direct contradiction to the other criteria but consideration of computer centre costing formulae suggests methods by which it may be met. At many computer centres such as that at the University of Waterloo, batch processing costs increase at a rate greater than the square of the core memory used and a large portion of the run cost of certain programs is simply due to this factor.

If the use of large data arrays can be avoided it should be possible to develop a very complex subcatchment hydrologic model and still incur computer costs less than those experienced by users of large core programs such as the U.S. E.P.A. SWMM model.

If the memory requirements can be kept down to about 30 kilobytes then a further possibility for meeting criterion number 9 presents itself. A number of micro computers are currently coming on the market equipped with the standard FORTRAN language and which are well within the price range of almost any engineering company or government agency. If the subcatchment hydrologic model could run efficiently on such a machine then the perceived cost of applying even a very complex model would be essentially limited to data collection.

SOURCES OF RUNOFF AND RUNOFF MECHANISMS

The study of runoff mechanisms and sources of runoff is fundamental to the construction of a hydrologic simulation model for such a model simply consists of a series of mathematical representations of those mechanisms which are felt to be significant.

A literature search revealed that there are numerous viable runoff mechanisms such as:

- impervious runoff
- pervious runoff
- runoff from channel precipitation
- runoff from agricultural tile drains
- groundwater flow

It was obvious that the relative importance of these would vary from basin to basin and from storm to storm. Thus in order to be generally applicable to almost any basin, a subcatchment hydrologic model should not be overly biased towards any of the mechanisms which are believed to be significant. Rather the model should allow for the potential occurrence of all significant runoff mechanisms.

In order to reflect this vein of thought, it was felt that the subcatchment hydrologic model should have the following features:

- the subcatchment should immediately be broken into pervious, impervious and channel areas since each of these three responds differently to rainfall
- groundwater flow should not be assumed to be either insignificant or constant
- tile drain flow should be included in an explicit manner
- expansion and contraction of channel and wetland areas should be allowed for
- the model should explicitly allow for the fact that some but not necessarily all of roof runoff may drain to pervious area and so be subject to infiltration
- the model should include the Hortonion concept of overland flow but should also be capable of modelling the development of a perched zone of saturation within the soil and even the fact that the water table itself may rise to the ground surface
- seasonal variations in the hydrologic characteristics of a catchment should be allowed for in a consistent manner.

The components of the hydrologic cycle which are included in the subcatchment hydrologic model are illustrated in Figure 1.

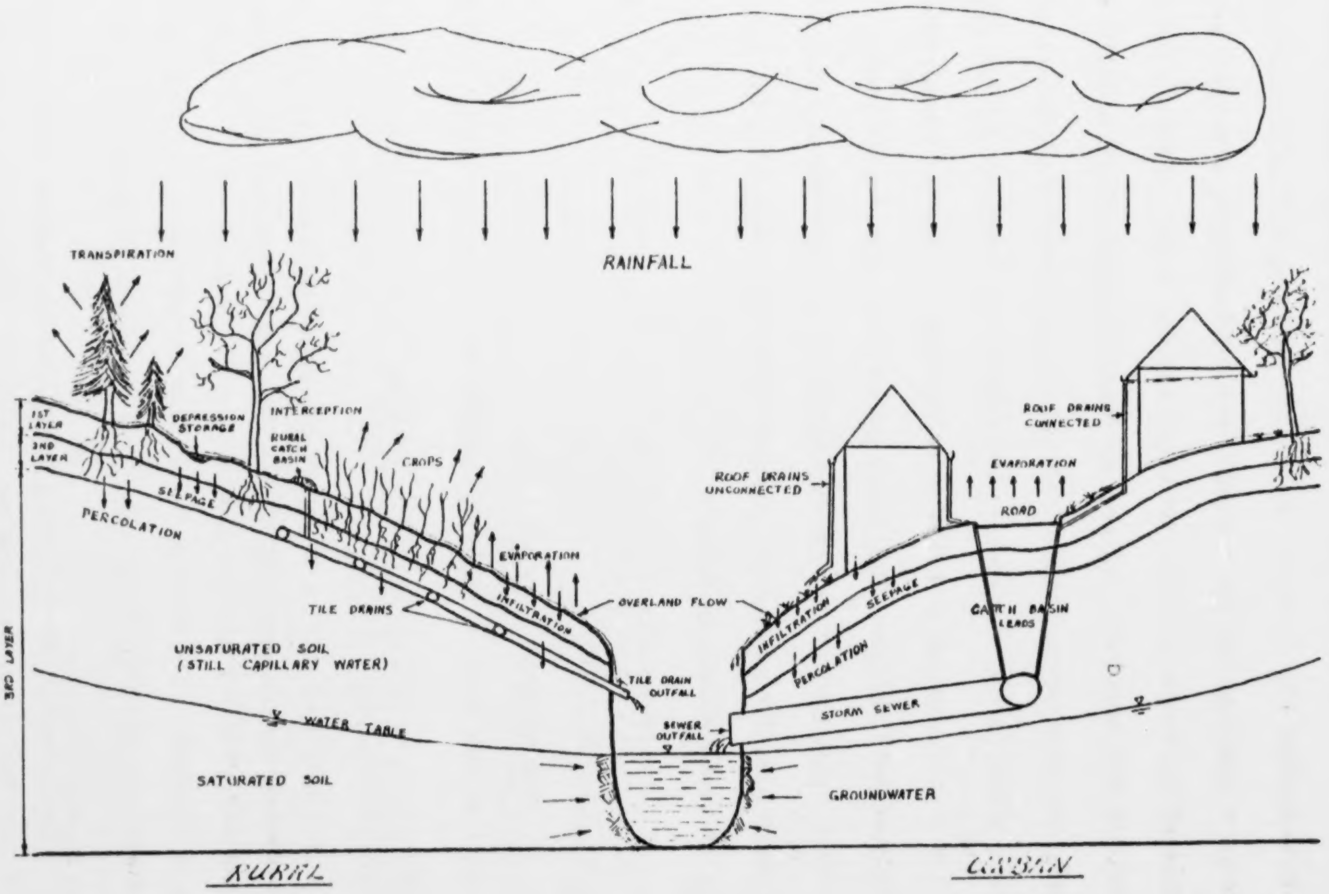
TRANSPIRATION
RAINFALL
1ST LAYER
2ND LAYER
3RD LAYER
DEPRESSION STORAGE
INTERCEPTION
RURAL CATCH BASIN
CROPS
SEEPAGE
PERCOLATION
EVAPORATION
TILE DRAINS
INFILTRATION
OVERLAND FLOW
UNSATURATED SOIL (STILL CAPILLARY WATER)
TILE DRAIN OUTFALL
WATER TABLE
SEWER OUTFALL
SATURATED SOIL
ROOF DRAINS UNCONNECTED
ROOF DRAINS CONNECTED
EVAPORATION
ROAD
INFILTRATION
SEEPAGE
PERCOLATION
CATCH BASIN LEADS
STORM SEWER
GROUNDWATER
RURAL
URBAN

FIGURE NO. 1 : COMPONENTS OF THE HYDROLOGIC CYCLE SIMULATED IN "SUBHYD"

DESCRIPTION OF SUBCATCHMENT HYDROLOGIC MODEL (SUBHYD)

Numerous concepts and methods of modelling the various components of the hydrologic cycle illustrated in Figure 1 were considered and a description of these may be found in reference 14. To some degree the resulting subcatchment model (SUBHYD) represents a marriage of the concepts and methodologies embodied in two previous sophisticated hydrologic models, one urban and one rural. (SWMM[2] and USDAHL-74[8]) SUBHYD also has numerous features not found in either SWMM or USDAHL-74 such as tile drain flow, interception, roof drains discharging to lawns, fully separated pervious and impervious areas, partial area concept and exponential depression storage equations.

The organization of, and flow of calculations within, the subcatchment hydrologic model is illustrated in Figure 2 which is a general flow chart for the model.

From Figure 2 it can be seen that within the model a subcatchment is immediately broken down into three sub-areas, each of which has a somewhat different response to rainfall. This subdivision is as follows:

- impervious area
- pervious area
- channel area

Each of the areas is generally viewed as a separate entity with the following exceptions:

- A user specified portion of impervious area is allowed to drain onto pervious area, bypassing interception. (RUNON) This was included so that the effect of roof drains discharging onto grassed areas could be simulated. This feature also allows the simulation of a parking lot draining to pervious area rather than to a catch basin.

- Channel area is treated as being dynamic and is allowed to expand and contract at the expense of pervious area during a storm. This feature allows the inclusion, to some degree, of the partial area concept which states that runoff comes from relatively small and constant saturated areas near channels.

FLOW CHART: SUBCATCHMENT HYDROLOGIC MODEL (SUBHYD)

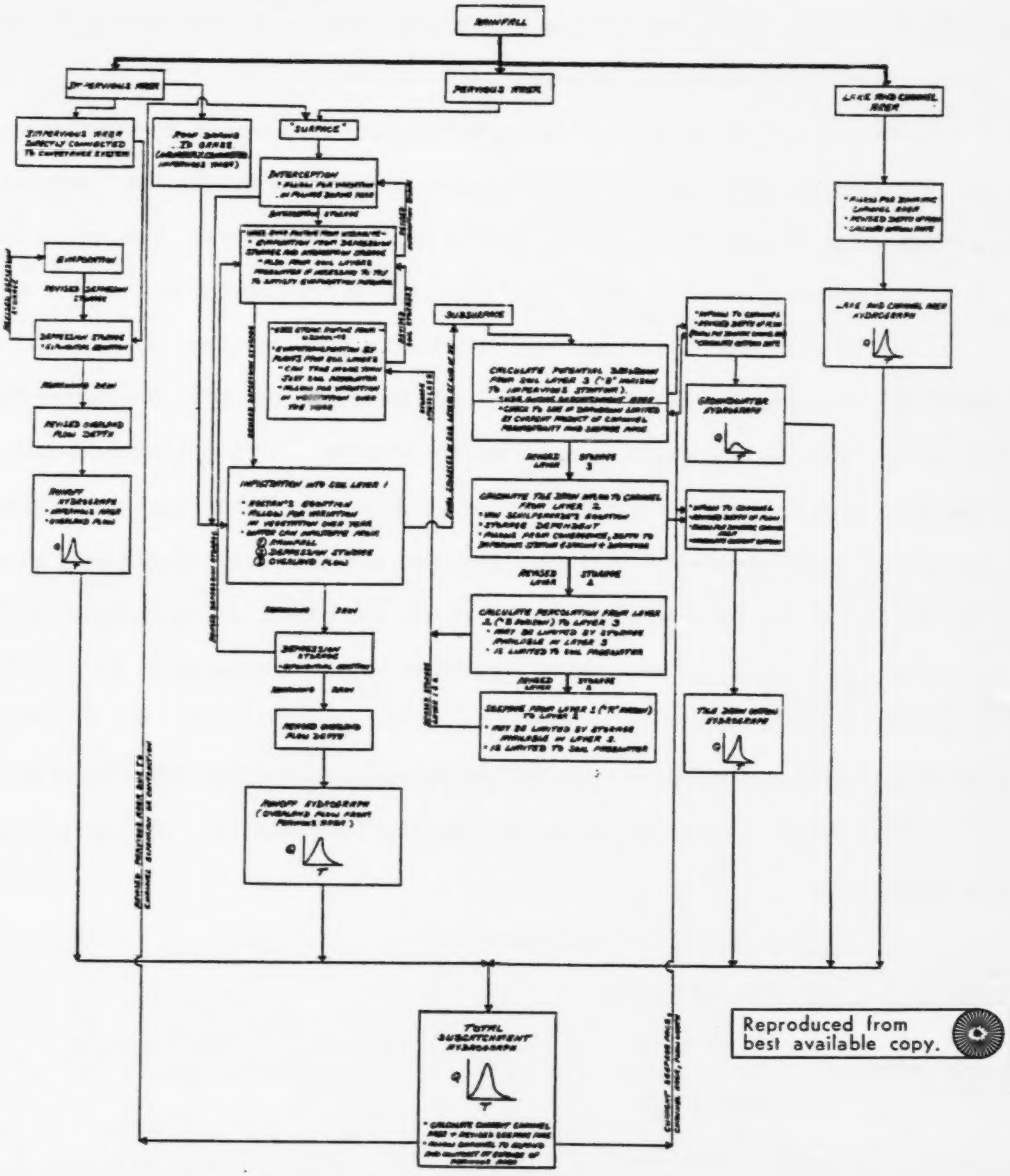
RAINFALL
IMPERVIOUS AREA
PERVIOUS AREA
LAKE AND CHANNEL AREA
IMPERVIOUS AREA DIRECTLY CONNECTED TO CONVEYANCE SYSTEM
ROOF DRAINS TO GRASS
"SURFACE"
INTERCEPTION
INTERCEPTION STORAGE
EVAPORATION
REVISED DEPRESSION STORAGE
DEPRESSION STORAGE
REMAINING RAIN
REVISED OVERLAND FLOW DEPTH
RUNOFF HYDROGRAPH
• IMPERVIOUS AREA
• OVERLAND FLOW
SUBSURFACE
INFILTRATION INTO SOIL LAYER 1
• HORTON'S EQUATION
• ALLOWS FOR VARIATION IN VEGETATION OVER YEAR
• WATER CAN INFILTRATE FROM
① RAINFALL
② DEPRESSION STORAGE
③ OVERLAND FLOW
RUNOFF HYDROGRAPH (OVERLAND FLOW FROM PERVIOUS AREA)
CALCULATE POTENTIAL DRAWDOWN FROM SOIL LAYER 3 ("B" HORIZON) TO IMPERVIOUS STRATUM).
• CHECK TO SEE IF DRAWDOWN LIMITED BY CURRENT PRODUCT OF CHANNEL PERMEABILITY AND SEEPAGE AREA
REVISED LAYER
STORAGE 3
CALCULATE TILE DRAIN INFLOW TO CHANNEL FROM LAYER 2.
• USE SCHILFGAARDE'S EQUATION
• STORAGE DEPENDENT
REVISED LAYER
STORAGE 2
CALCULATE PERCOLATION FROM LAYER 2 ("B HORIZON") TO LAYER 3
• MAY BE LIMITED BY STORAGE AVAILABLE IN LAYER 3
• IS LIMITED TO SOIL PARAMETER
SEEPAGE FROM LAYER 1 ("A" HORIZON) TO LAYER 2
• MAY BE LIMITED BY STORAGE AVAILABLE IN LAYER 2.
• IS LIMITED TO SOIL PARAMETER
• INFLOW TO CHANNEL
• REVISED DEPTH OF FLOW
GROUNDWATER HYDROGRAPH
TILE DRAIN INFLOW HYDROGRAPH
LAKE AND CHANNEL AREA HYDROGRAPH
TOTAL SUBCATCHMENT HYDROGRAPH
Q
T

FIGURE NO. 2 GENERAL FLOW CHART OF "SUBHYD"

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best available copy.

The model has been programmed in such a way that variable time steps may be used for computational economy during continuous simulations. During periods of rainfall when rapid changes occur in hydrologic processes the time steps should be fairly small in order to simulate sudden changes but once rainfall ends, time steps may be increased in order to achieve computational economy.

Throughout the model flow equations (infiltration, seepage, percolation, overland flow) are solved simultaneously with applicable continuity equations with the result that average rates of each process for each time step are determined. This means that if a flow is calculated for a one hour time step, this is an average flow which would be sustained for one hour. There may well have been a more instantaneous peak within that hour which was greater, but the model will only be able to simulate this shorter duration, higher intensity peak if smaller time steps are utilized. This averaging process has been found to make the model very stable and no convergence problems have arisen even when time steps somewhat greater than the travel time across a subcatchment were tried. Due to this fact choice of time step may be at the preference of the modeller and is not governed by stability of the model equations. For instance, even on a very small subcatchment, if a modeller is not interested in any peak flow which is not sustained for at least five minutes there is no need to specify a time step of less than five minutes.

The model calculates and prints out the following flow hydrographs for any subcatchment:

- runoff from channel precipitation
- impervious runoff
- pervious runoff
- tile drain outflow
- groundwater flow

These are added to give the total flow from the subcatchment and comparisons are made to observed flow hydrographs if such are available.

The subcatchment hydrologic model (SUBHYD) was programmed in the FORTRAN IV computer language on an Applied Digital Data Systems System 70 (ADDS-70) micro computer.

The program as written has no practical limits other than disk storage capacity for data, on either the number of subcatchments or time steps that may be analyzed in a given run. A typical run for a single subcatchment requires about 2 minutes from initiation to final printout. The program requires about 28.5 kilobytes of memory to run unoverlayed using the ADDS ADOS version 1.4 FORTRAN software. Since about 7 kilobytes of this is tied up in extensive format literal strings and a bi-level execution trace it is conceivable that the model could run with somewhat less than 28 kilobytes of memory without affecting the calculations.

SUBHYD has also been interfaced with the Storm Water Management Model Transport Block in the following manner.

Subcatchment hydrographs are generated by SUBHYD and then an interface program reformats these into SWMM TRANSPORT block input form and transmits them from the micro computer to the Multiple Access computer system in Don Mills, Ontario. Here they become part of a standard SWMM data set and detailed routing calculations are performed. This forms part of a sophisticated major-minor drainage analysis methodology in which inlet control is simulated and simultaneous routing calculations performed for both the major and minor drainage system for any storm. (Thompson and Dennison, 1979[15])

VALIDATION OF SUBCATCHMENT HYDROLOGIC MODEL

Introduction

The model was tested at a subcatchment level for both urban and rural discrete simulations. Both SWMM,[2] a well known and accepted urban hydrologic model and

HYMO[20] a well known rural hydrologic model were also used on the same simulations for comparative purposes.

One parameter, which is not known for validation simulations and which would be a design parameter for design simulations, is initial soil moisture content. This parameter was varied within reasonable limits for each simulation until the simulated volume of runoff agreed closely with the observed volume of runoff. This procedure has been followed by other investigators including Kouwen as described by Gorrie[4]. By reasonable limits the following is meant:

> If the watershed has not received rainfall for several days during the summer season, it is to be expected that all the soil free water would have drained from the surface soil layers and some capillary stoarge would be coming available due to plant evapotranspiration. Thus if a watershed has not received rainfall for a week or more the initial soil moisture content should be such that all soil freewater storage and also some capillary storage should be available to infiltrating water.

Urban Validation

It was decided to test the Subcatchment Hydrologic Model (SUBHYD) on a catchment located in Northwest Waterloo very near to the University of Waterloo. This catchment was chosen because of its proximity to the University and also because it had previously been instrumented and the U.S. E.P.A. SWMM (1975) model applied to it.[17]

The area comprises 18.2 hectares (45 acres) of predominantly single family residential land use. It is a relatively high income area with hilly half acre lots and homes valued at approximately $100,000 each.[17]

The location of the Northwest Waterloo Test Catchment is shown in Figure 3.

The catchment data required for SUBHYD was abstracted from the report and background material produced by Vespi.[17] Vespi had broken the area down into 17 subcatchments and done detailed sewer routing using the SWMM (1975) transport block and time steps of about 5 minutes.

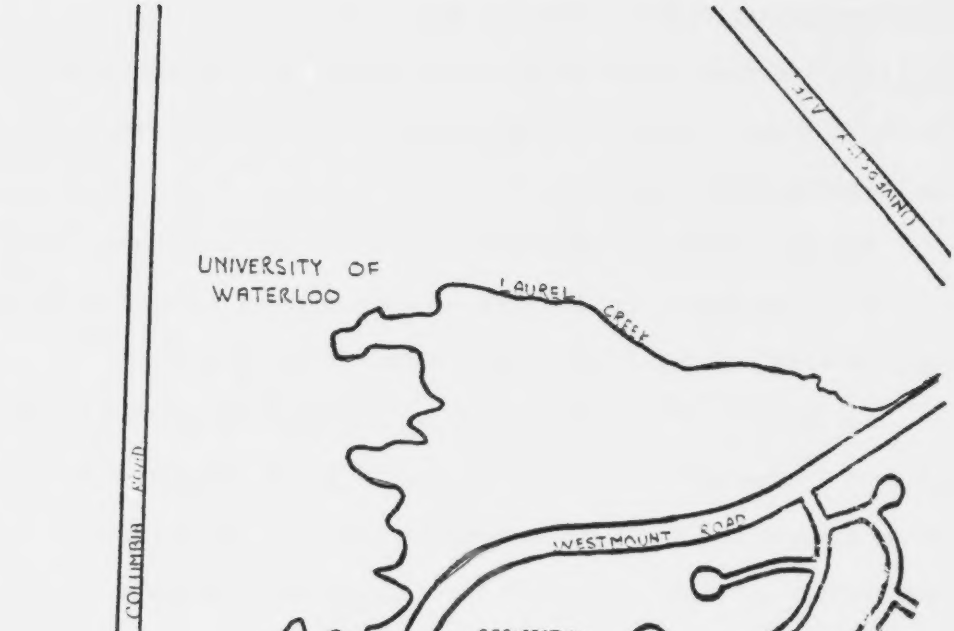

UNIVERSITY AVE
UNIVERSITY OF WATERLOO
LAUREL CREEK
COLUMBIA ROAD
WESTMOUNT ROAD
RESURECTION COLLEGE
OLD POST ROAD
ACADEMY
POST HORN
— APPROXIMATE CATCHMENT BOUNDARY

FIGURE NO. 3 : NORTHWEST WATERLOO URBAN TEST CATCHMENT
(after reference 17)

In order to provide a direct test of the overall subcatchment model and of the averaging type of algorithms used it was decided to simulate the area as a single subcatchment using SUBHYD with half hour time steps.

The maximum values of depression storage were estimated with the aid of the SWMM (1975) user's manual and the impervious area slope was determined as the weighted average sewer slope.

Soil parameters were determined with the aid of the Waterloo county soils report.

It was decided to test SUBHYD on a storm which occurred on November 10, 1977 since this was a well defined storm of 1.96 cm (0.77 in.) of rainfall.

For comparative purposes both the SWMM and the standard HYMO model[20] were also used to simulate the catchment response to the November 10 storm. All models treated the area as a single subcatchment and the U.S. Soil Conservation Service (S.C.S.) curve number in HYMO was adjusted so that calculated and observed volumes of runoff agreed.

A tabled comparison of the various model versus observed results may be found in Table 1 while graphical comparisons of the SUBHYD, SWMM and HYMO predicted hydrographs versus the observed hydrograph and each other may be found in Figures 4 to 6.

The SUBHYD simulation of this storm had the following statistics relative to the observed flows:

- error in peak flow = 2.32%
- error in volume of runoff = 1.88%
- correlation coefficient = 0.918

The previously mentioned detailed calibrated SWMM simulation of this runoff event performed by Vespi (1977) resulted in an error of about 15% in volume of runoff and about 12% in peak flow.

TABLE I: HYDROLOGIC MODEL COMPARISON

NORTHWEST WATERLOO TEST CATCHMENT - STORM OF NOVEMBER 10, 1977

1.96 CM RAINFALL (0.77 IN)

MODEL	MEAS. PEAK FLOW (CMS)	CALC. PEAK FLOW (CMS)	ERROR %	MEAS. VOL. (1000 M**3)	CALC. VOL. (1000 M**3)	ERROR %	MEAS. TIME TO PEAK (HR)	CALC. TIME TO PEAK (HR)	ERROR (HR)
HYMO (DT = 0.5 HR)	0.0345	0.0376	9.08	0.357	= 0.357	= 0	< 0.5	= 1.0	0.5
SWMM (DT = 0.5 HR)	0.0345	0.0370	7.25	0.357	0.43	20.4	< 0.5	0.5	≈ 0.5
SUBHYD (DT = 0.5 HR)	0.0345	0.0337	-2.32	0.357	0.363	1.88	< 0.5	< 0.5	0

NOTE:
1) 1 CMS = 35.3 CFS
2) 1000 M**3 = 0.81 ACRE FT.
3) DRAINAGE AREA = 0.182 SQ. KM
= 0.0695 SQ. MI.
= 45 ACRES

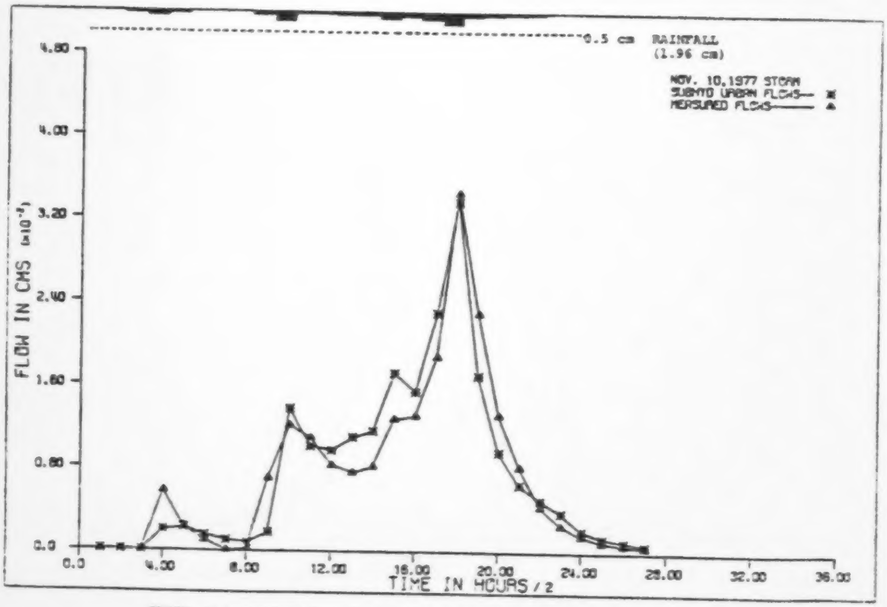


FIGURE 4 : NOVEMBER 10, 1977 STORM - SUBHYD VERSUS OBSERVED FLOWS

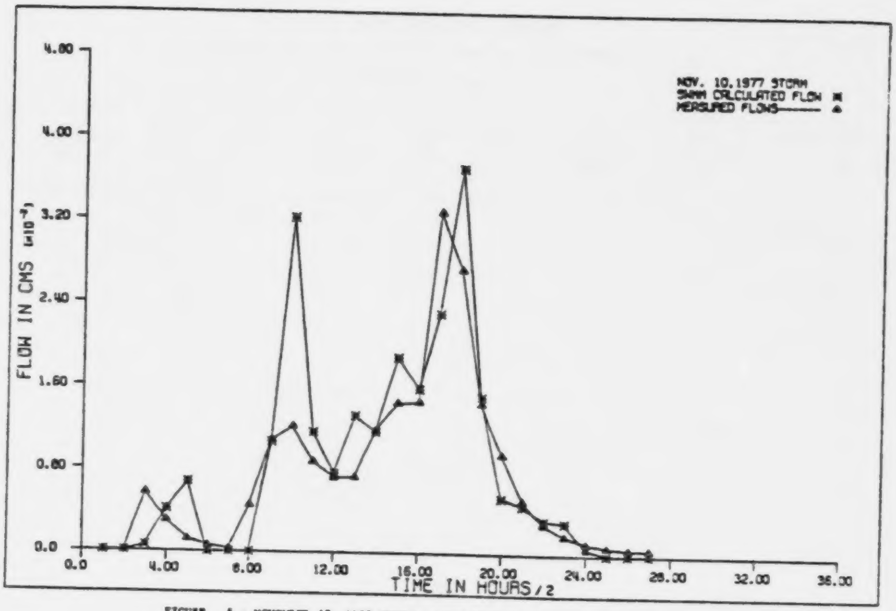


FIGURE 5 : NOVEMBER 10, 1977 STORM - SWMM VERSUS OBSERVED FLOWS

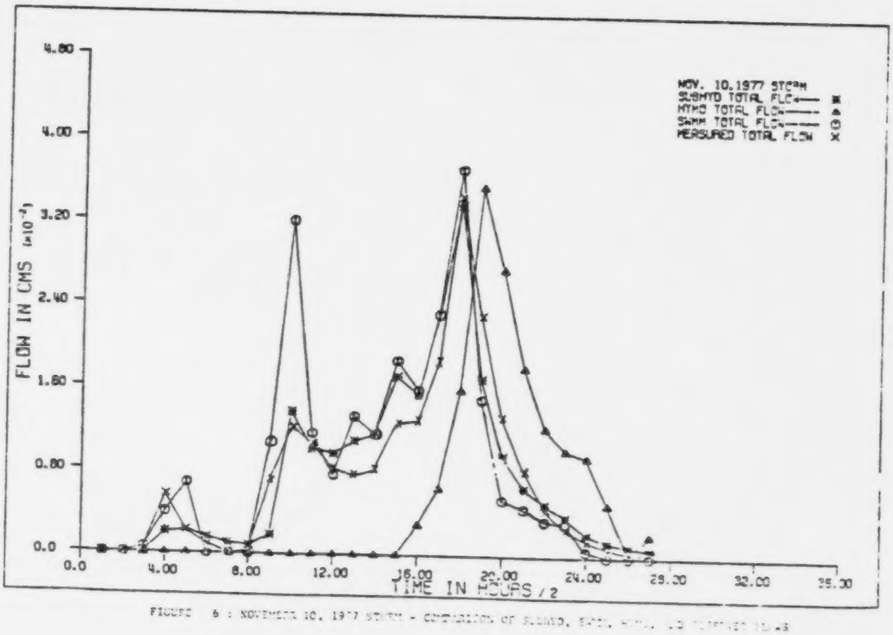


FIGURE 6 : NOVEMBER 10, 1977 STORM - COMPARISON OF SUBHYD, SWMM, HYMO, AND MEASURED FLOWS

Rural Validation

The West Canagagigue Creek watershed above Floradale, Ontario was chosen as a test rural subcatchment for a number of reasons:

1. its close promitity to the University of Waterloo
2. it has been gauged by the Water Survey of Canada
3. it has been considered to be a very typical agricultural watershed[3]
4. it was previously chosen as a test catchment by the Pollution from Land Use Activities Reference Group (PLUARG) so that a great deal of data was available for it.

Some of the data collected for this basin by PLUARG researchers included a farm by farm tile drain survey,[19] cross sections,[19] rain gauge data, and detailed land use information.[3]

The West Canagagigue catchment of about 19.46 sq. km. (4,800 acres or 7.5 sq. mi.) is referred to as AG-4 in the various PLUARG reports and represents the watershed of Canagagigue Creek, a tributary of the Grand River which flows into Lake Erie. The catchment is located in Peel Township, Wellington County, about 22 km north of Waterloo. The topography of the area is gently undulating and the main soil type is loam with small areas of muck and river bottom flat lands.

The main agricultural activity is mixed farming[3] and predominantly dariy with lesser numbers of beef and swine. Crops grown include corn for silage, small grains and hay. A large number of the farmers are traditional old order Mennonites, however their farming practices are very modern and they utilize extensive tile drain networks.

The extent of the West Canagagigue test subcatchment is illustrated in Figure 7.

Due to the large amount of documented data available for the catchment it was possible to abstract values for most of the various parameters directly from published sources.

Hourly streamflow values for storms of interest were obtained from the Water Survey of Canada from records of their station number 02GA036.

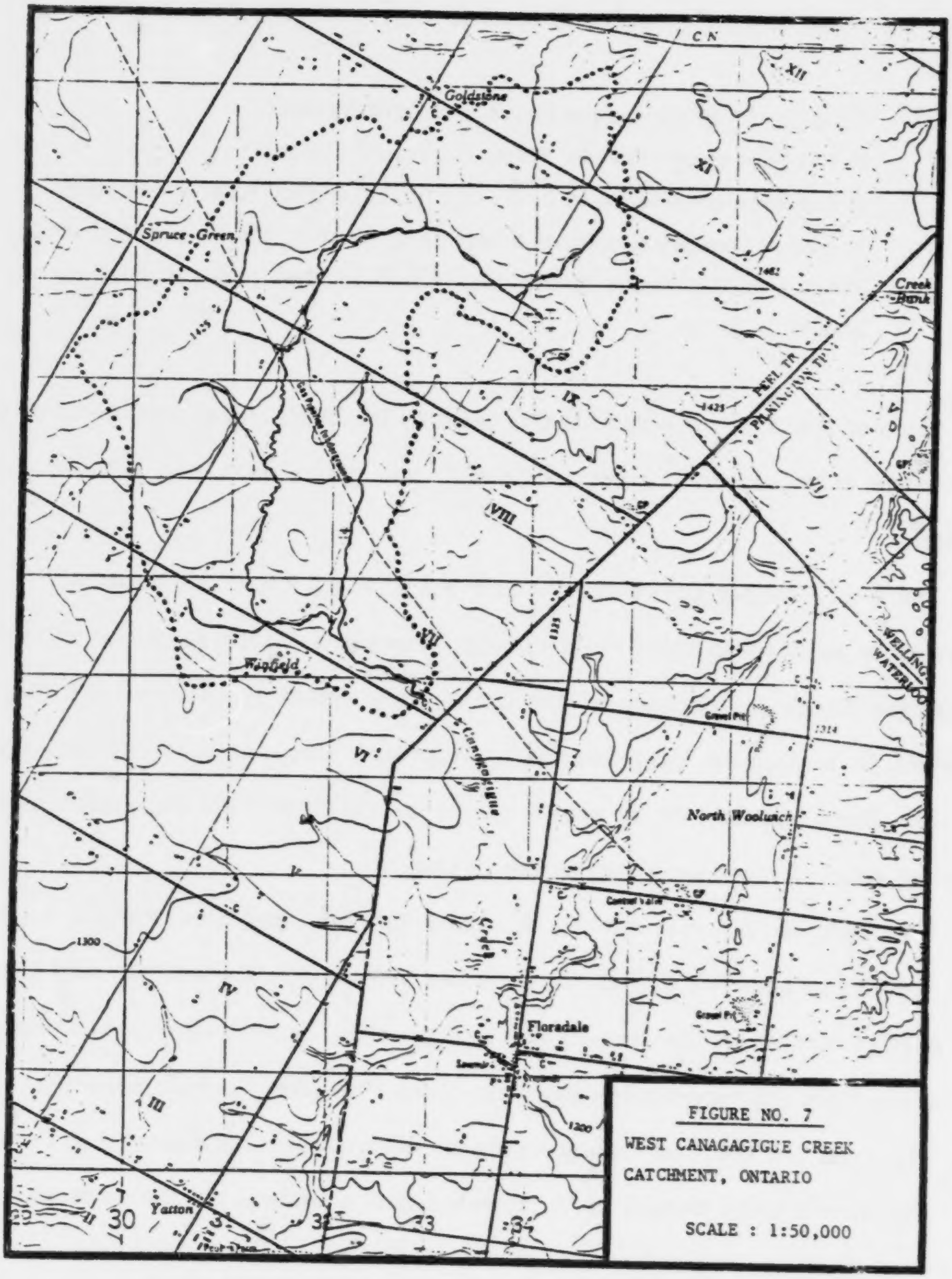

CN
XII
Goldstone
XI
Spruce Green
Creek Bank
1462
1421
IX
PEEL TP.
PILKINGTON TP.
VII
VIII
VII
Winfield
VI
North Woolwich
Gravel Pit
WELLINGTON
WATERLOO
V
1300
IV
Floradale
Gravel Pit
III
1200
II
Yatton
30
31
32
33
34

FIGURE NO. 7

WEST CANAGAGIGUE CREEK CATCHMENT, ONTARIO

SCALE : 1:50,000

Rain gauge data in hourly increments was obtained from the PLUARG rain gauge maintained by Dr. Anderson of the University of Windsor.

Landuse information was abstracted from the PLUARG landuse report on the catchment prepared by Frank and Ripley, 1977 which was based upon data collected in 1975-76.

Soil data including depths and textural classifications of each soil layer were determined from the Wellington County Soil Survey.

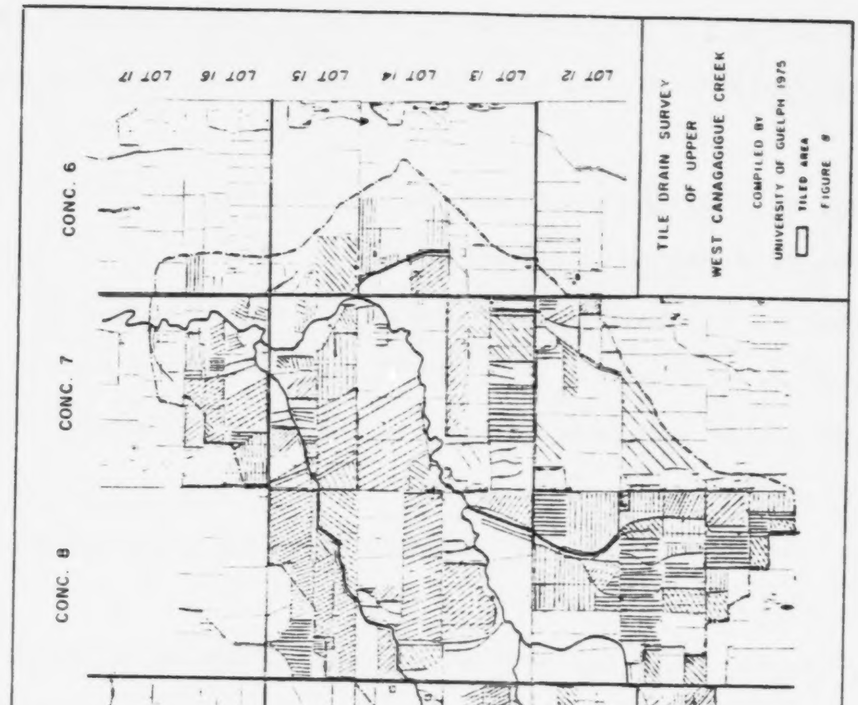
Tile drain data was determined with the aid of a detailed farm by farm tile drain survey carried out by Whiteley et al, 1976[19] for PLUARG. Figure 8 is a composite of the information gathered for each farm and serves to illustrate the extent and complexity of the tile drain network in existance as of 1976. Forty percent of the catchment was systematically tiled and Whiteley et al (1976) found no strong correlation between tiling and soil type.

Pan evaporation data was obtained from the University of Guelph for their evaporation pan at nearby Elora. Temperature data was also obtained from this source and used as input to the LANUSE routine of USDAHL-75 (a metric version of USDAHL-74). This routine was used to determine growth indices for each landuse and week throughout the period of interest and thereby allow for seasonal variations in vegetation.

The remaining data concerning drainage area, flow lengths and slopes were estimated from the 1:50,000 scale topographic map of the catchment.

The initial soil moisture content was varied within reasonable limits as described previously in order to match observed peak flow and volume of runoff. This was done because this is one parameter which was not known for validation purposes and which would be a design parameter for design purposes.

The Subcatchment Hydrologic Model (SUBHYD) was tested on three storm runoff events which occurred on the West Canagagigue catchment during the period extending from the beginning of May to the end of June, 1976. These storms differed widely in rainfall amount, intensity, duration and antecedent moisture conditions.


CONC. 6
CONC. 7
CONC. 8
CONC. 9
CONC. 10
CONC 11
LOT 12
LOT 13
LOT 14
LOT 15
LOT 16
LOT 17
☐ TILED AREA

TILE DRAIN SURVEY
OF UPPER
WEST CANAGAGIGUE CREEK

COMPILED BY
UNIVERSITY OF GUELPH 1975

FIGURE 8

All SUBHYD parameters were held constant for these storms with the following exceptions:

- meterological input (rainfall, pan evaporation)
- growth index values (from growth index curves determined by LANUSE routine of USDAHL-75)
- initial soil moisture content (varied within reasonable limits)
- Manning's "n" for the channel (this was allowed to increase from 0.025 in the spring to 0.050 in mid-summer when the channel is somewhat overgrown).

The HYMO and the SWMM hydrologic models were also applied to this catchment for each storm for comparative purposes. The S.C.S. curve number in HYMO was varied to match observed volume of runoff since a similar operation was done for SUBHYD by varying the initial mositure content. The SWMM infiltration parameters were not varied since this model has not normally responded significantly to changes in these parameters.[12]

Simulation of May 5-7, 1976 Storm

This storm was essentially a drizzel that continued for more than one day. Although 2.9 cm. (1.14 in.) of rain fell, the maximum one hour intensity was only 0.2 cm/hr. (0.079 in/hr).

SUBHYD simulated the observed runoff hydrograph with the following statistics:

- error in peak flow = 2.3%
- error in volume of runoff = 7.0%
- correlation coefficient = 0.905

A tabled comparison of the various model versus observed results may be found in Table 2 while graphical comparisons of the SUBHYD, SWMM and HYMO predicted hydrographs versus the observed hydrograph and each other may be found in Figures 9 to 12.

TABLE 2: HYDROLOGIC MODEL COMPARISON

WEST CANAGAGIGUE CREEK - STORM OF MAY 5-7, 1976

2.9 CM (1.14 IN) RAINFALL

MODEL	MEAS. PEAK FLOW (CMS)	CALC. PEAK FLOW (CMS)	ERROR %	MEAS. VOL. (1000 M**3)	CALC. VOL. (1000 M**3)	ERROR %	MEAS. TIME TO PEAK (HR)	CALC. TIME TO PEAK (HR)	ERROR (HR)
HYMO (DT = 0.5 HR)	4.28	5.06	18.2	283	≈283	≈0	3	4	1
SWMM (DT = 0.5 HR)	4.28	0.273	-93.6	283	14.0	-95.1	3	5	2
SUBHYD (DT = 1.0 HR)	4.28	4.18	-2.34	283	303	7.07	3	6	3

NOTE:
1) 1 CMS = 35.3 CFS
2) 1000 M**3 = 0.81 ACRE FT.
3) DRAINAGE AREA = 19.46 SQ. KM
= 7.51 SQ. MI.
= 4809 ACRES

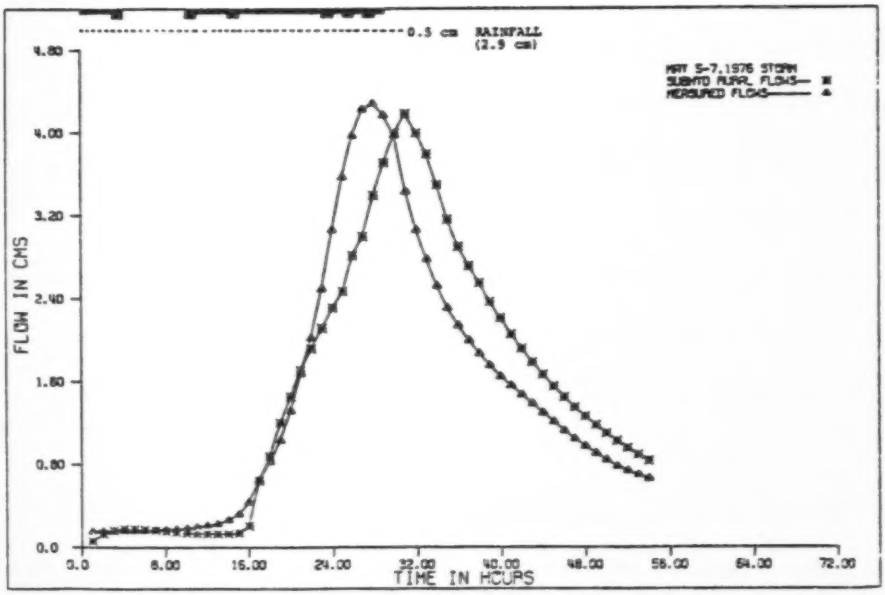


FIGURE 9 : MAY 5-7 STORM - SUBHYD VERSUS OBSERVED FLOWS

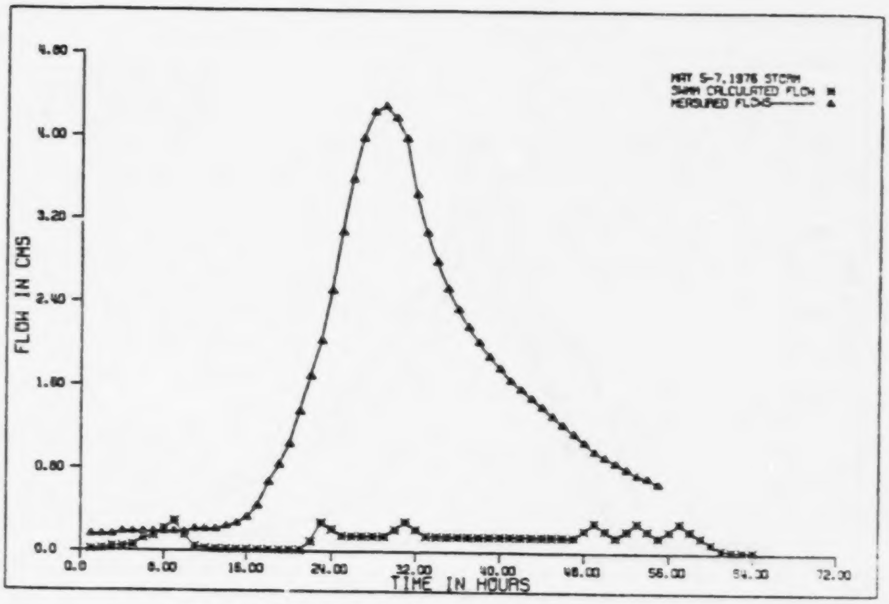


FIGURE 10 : MAY 5-7 STORM - SWMM VERSUS OBSERVED FLOWS

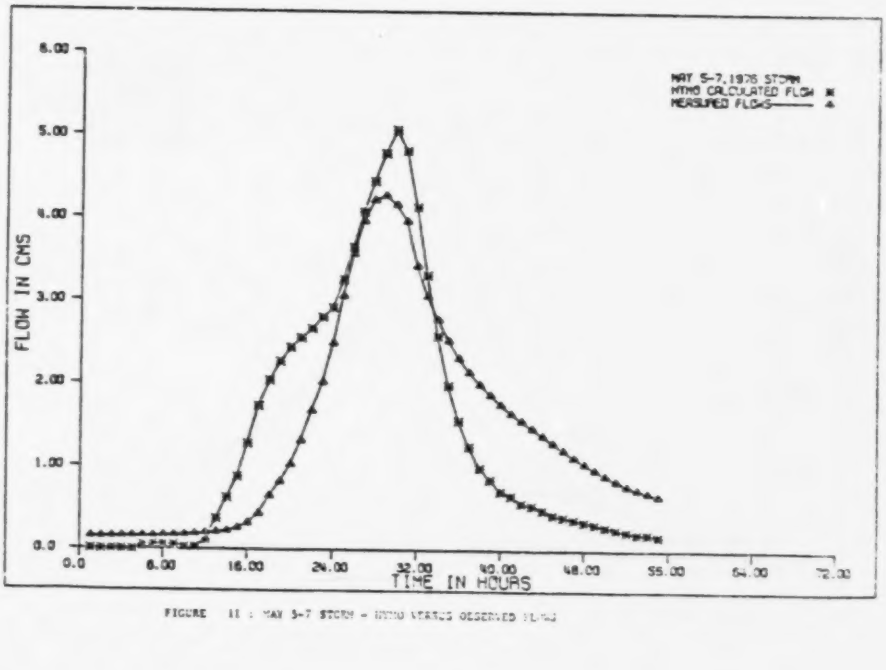


FIGURE 11 : MAY 5-7 STORM - HYMO VERSUS OBSERVED FLOWS

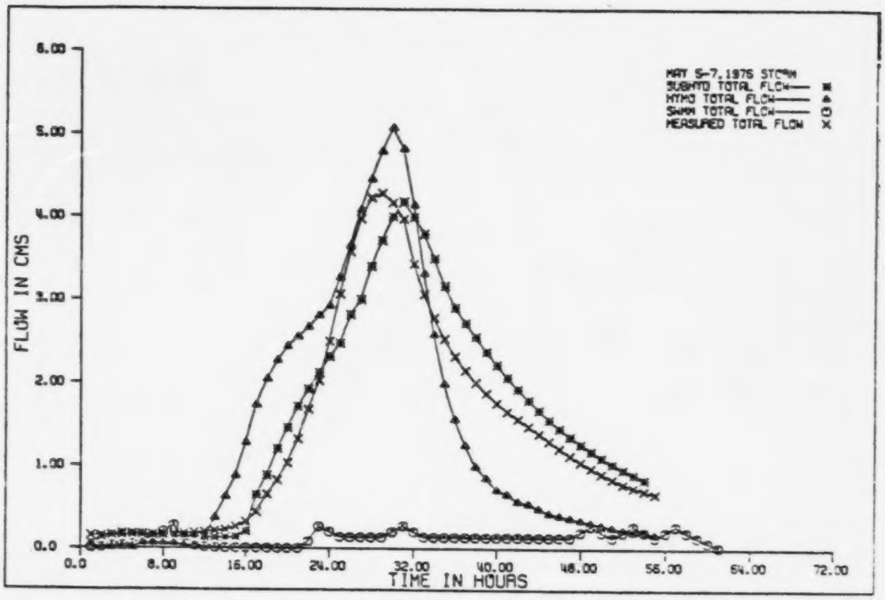


FIGURE 12 : MAY 5-7 STORM - COMPARISON OF SUBHYD, SWMM, HYMO, AND OBSERVED FLOWS

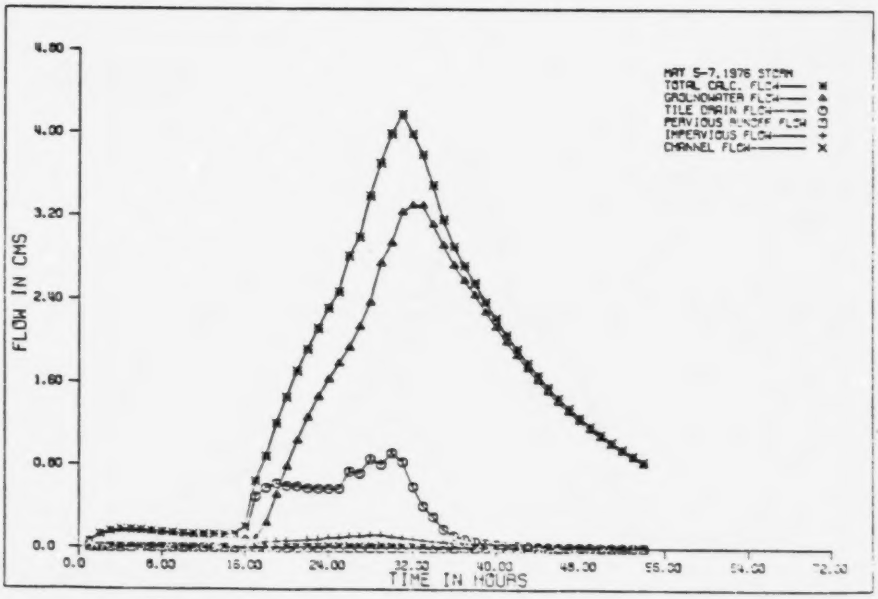


FIGURE 13 MAY 5-7 STORM - SUBHYD BREAKDOWN OF FLOW SERIES

The breakdown of the sources of runoff for this runoff event, as determined by SUBHYD may be found in Figure 13. The SUBHYD results indicate that this storm event was dominated by groundwater flow with contributions from tile drains, impervious area, channel precipitation and pervious area in decreasing order of importance.

Simulation of June 13, 1976 Storm

This storm occurred after almost a month without rain, and the catchment was so dry that the creek had been dried up for over a week. This was a very short intense storm of 3.1 cm (1.22 in.) with a maximum one hour intesity of 1.8 cm/hr. (0.71 in/hr).

SUBHYD simulated this storm as shown in Figure 14 with the following statistics:

- error in peak flow = 0.47%
- error in volume or runoff = 3.9%
- correlation coefficient = 0.873

Tabled and graphical comparisons of the various models to each other and observed flows may be found in Table 3 and Figures 14 to 17.

The sources of runoff during this storm, as determined by SUBHYD are illustrated in Figure 18. This runoff event was dominated by tile drain flow with additional contributions from impervious area, groundwater, channel precipitation and pervious area in decreasing order of significance. Tile drain flow was so important during this storm because the high rainfall intensities involved were able to build up a significant head on the tiles thus causing them to flow at greater rates. The very low intensities experienced during the earlier May 5-7 storm did not build up a significant head on the tiles with the result that most of the water during the May storm was able to percolate through to groundwater.

Simulation of June 30, 1976 Storm

The characteristics of this storm lay somewhat between those of the two previously discussed storms, with the exception of total rainfall.

TABLE 3: HYDROLOGIC MODEL COMPARISON

WEST CANAGAGIGUE CREEK - STORM OF JUNE 13, 1976

3.1 CM (1.22 IN) RAINFALL

MODEL	MEAS. PEAK FLOW (CMS)	CALC. PEAK FLOW (CMS)	ERROR %	MEAS. VOL. (1000 M**3)	CALC. VOL. (1000 M**3)	ERROR %	MEAS. TIME TO PEAK (HR)	CALC. TIME TO PEAK (HR)	ERROR (HR)
HYMO (DT = 0.5 HR)	3.71	1.87	-49.6	500	≈ 500	≈ 0	2	4	2
SWMM (DT = 0.5 HR)	3.71	2.68	-27.8	500	15.1	-96.98	2	2	0
SUBHYD (DT = 1.0 HR)	3.71	3.73	0.47	500	519	3.91	2	2	0

NOTE:
1) 1 CMS = 35.3 CFS
2) 1000 M**3 = 0.81 ACRE FT.
3) DRAINAGE AREA = 19.46 SQ. KM
= 7.51 SQ. MI.
= 4809 ACRES

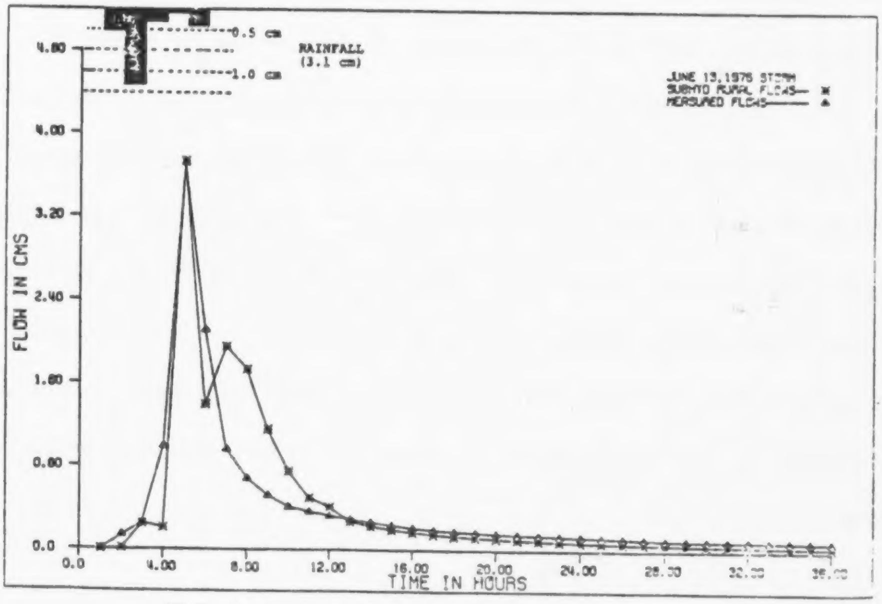


FIGURE 14 : JUNE 13 STORM - SUBHYD VERSUS OBSERVED FLOWS

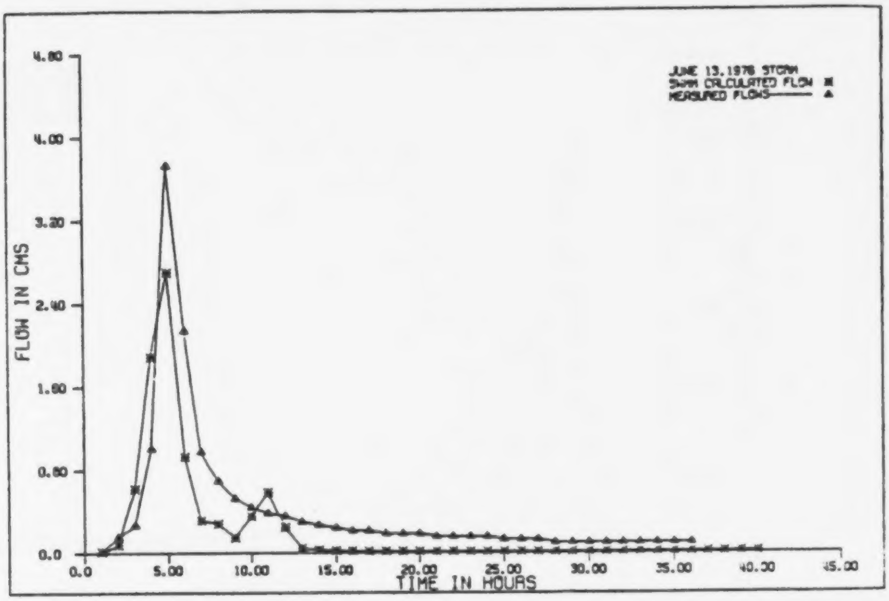


FIGURE 15 : JUNE 13 STORM - SWMM VERSUS OBSERVED FLOWS

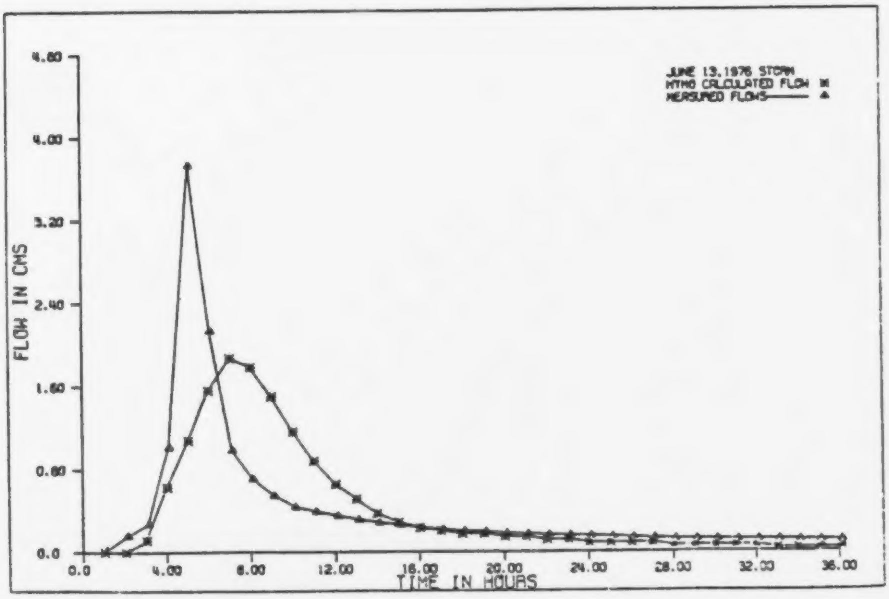


FIGURE 16 : JUNE 13 STORM - HYMO VERSUS OBSERVED FLOWS

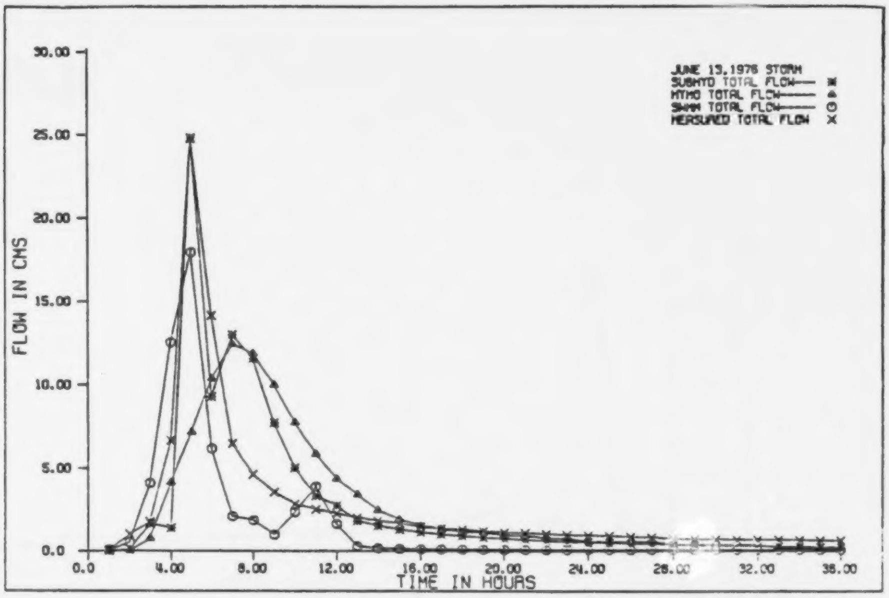


FIGURE 17 : JUNE 13 STORM - COMPARISON OF SUBHYD, SWMM, HYMO, AND OBSERVED FLOWS

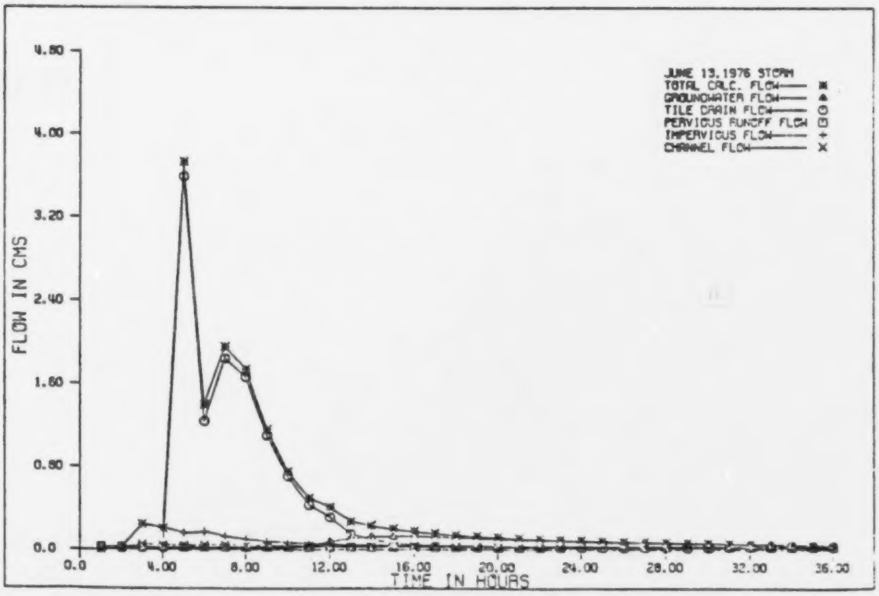


FIGURE 18 : JUNE 13 STORM - SUBHYD BREAKDOWN OF FLOW SOURCES

5.2 cm (2.05 in.) of rainfall fell at a maximum one hour intensity of 1.4 cm/hr. (0.55 in/hr.). The catchment was dry but not as dry as it had been before the June 13 storm since the creek had just barely dried up.

The statistics of the comparison of the SUBHYD simulation and the observed hydrograph were as follows:

- error in peak flow = 0.70%
- error in volume or runoff = 0.59%
- correlation coefficient = 0.872

Comparisons of the various models to each other and observed flows may be found in Table 4 and Figures 19 to 22.

Figure 23 illustrates the sources of runoff during this storm as determined by SUBHYD. This runoff event was dominated by tile drain and groundwater flow. The tile drains contributed more to the peak flow but groundwater flow produced a greater volume of runoff. Smaller amounts of flow were produced by the mechanisms of impervious runoff, channel precipitation and pervious runoff in decreasing order of importance.

This storm had high intensities of rainfall although not as high as were experienced on June 13, 1976, and this caused a significant head to be built up on the tiles creating substantial tile drain flow.

Effect of Tile Drainage on Peak Runoff

In recent years farmers in the Canagagigue and other watersheds have been installing extensive tile drainage systems. These have been installed so that fields will dry out more rapidly in the spring thereby allowing farmers to begin working their fields about two weeks earlier than previously.[19]

By intent these tile drainage systems must contribute to spring runoff and at the outset of this work it was believed possible that they might contribute significantly to storm runoff as well.

TABLE 4: HYDROLOGIC MODEL COMPARISON

WEST CANAGAGIGUE CREEK - STORM OF JUNE 30, 1976

3.2 CM (2.05 IN) RAINFALL

MODEL	MEAS. PEAK FLOW (CMS)	CALC. PEAK FLOW (CMS)	ERROR %	MEAS. VOL. (1000 M**3)	CALC. VOL. (1000 M**3)	ERROR %	MEAS. TIME TO PEAK (HR)	CALC. TIME TO PEAK (HR)	ERRO (HR)
HYMO (DT = 0.5 HR)	4.53	6.44	42.2	174	≈174	≈0	2	3	1
SWMM (DT = 0.5 HR)	4.53	2.08	-54.1	174	24.70	-85.8	2	18	16
SUBHYD (DT = 1.0 HR)	4.53	4.50	-0.70	174	173	-0.59	2	3	1

NOTE: 1) 1 CMS = 35.3 CFS
2) 1000 M**3 = 0.81 ACRE FT.
3) DRAINAGE AREA = 19.46 SQ. KM
= 7.51 SQ. MI.
= 4809 ACRES

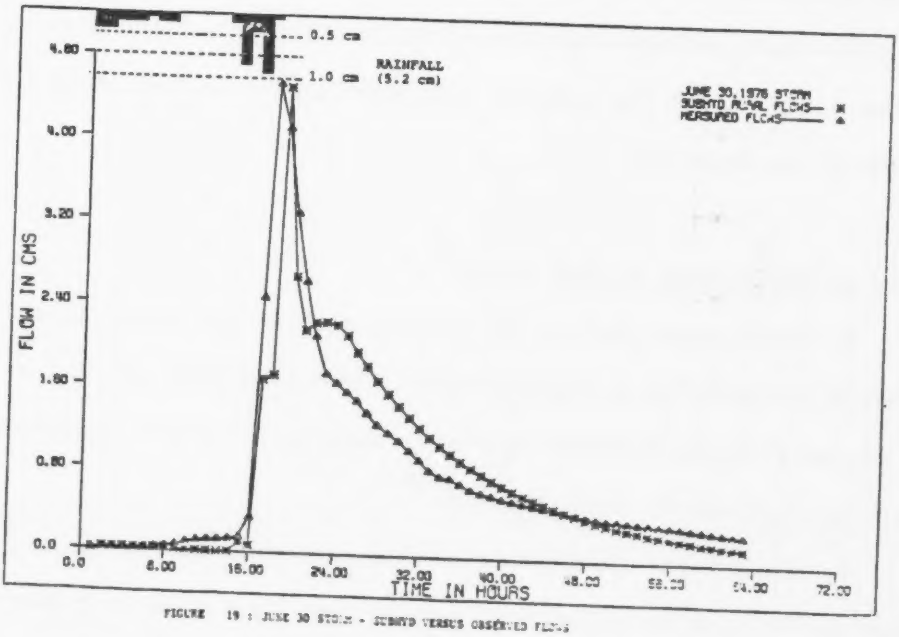


FIGURE 19 : JUNE 30 STORM - SUBHYD VERSUS OBSERVED FLOWS

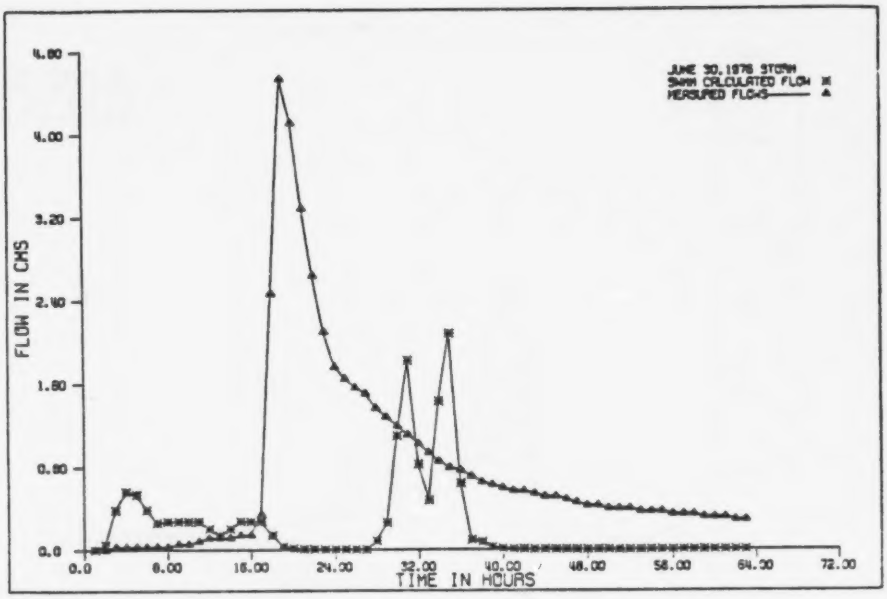


FIGURE 20 : JUNE 30 STORM - SWMM VERSUS OBSERVED FLOWS

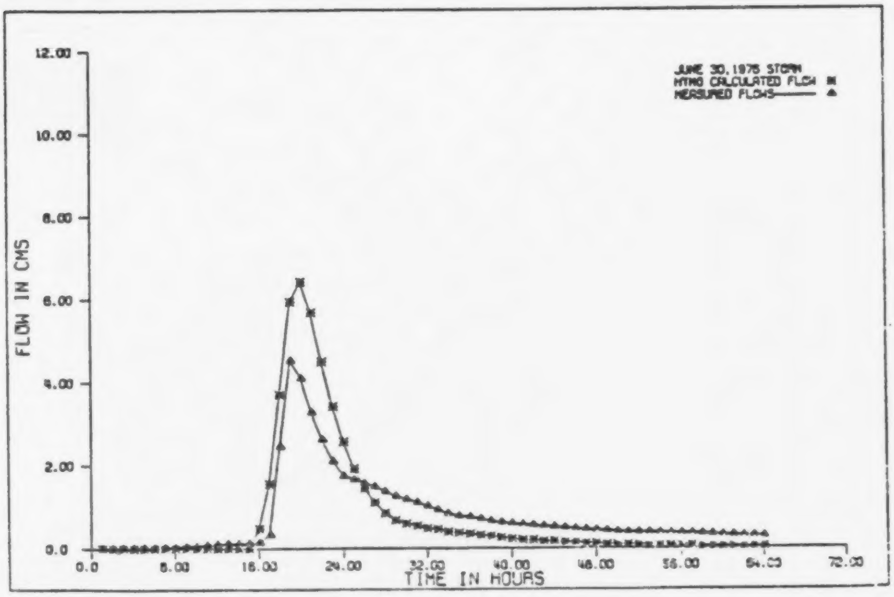


FIGURE 21 : JUNE 30 STORM - HYMO VERSUS OBSERVED FLOWS

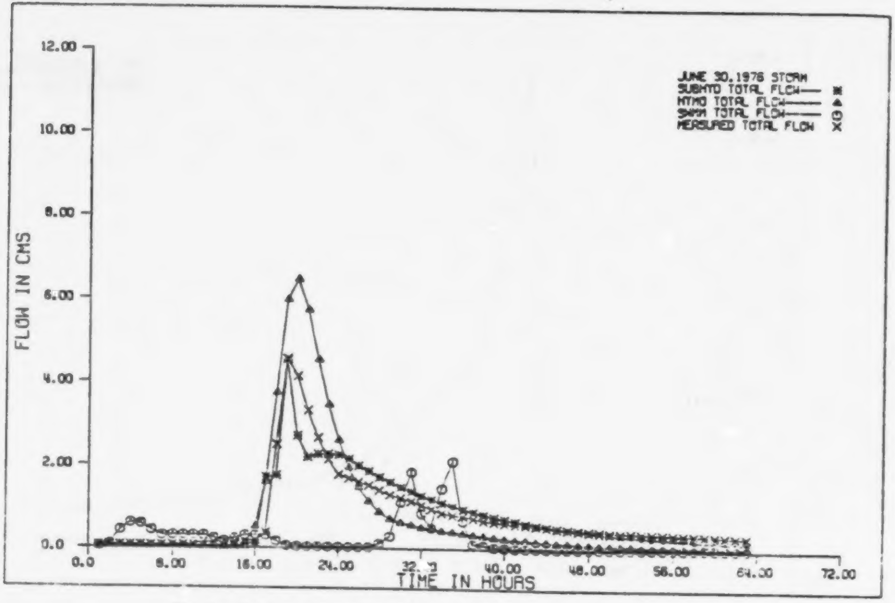


FIGURE 22 : JUNE 30 STORM - COMPARISON OF SUBHYD, SWMM, HYMO, AND OBSERVED FLOWS

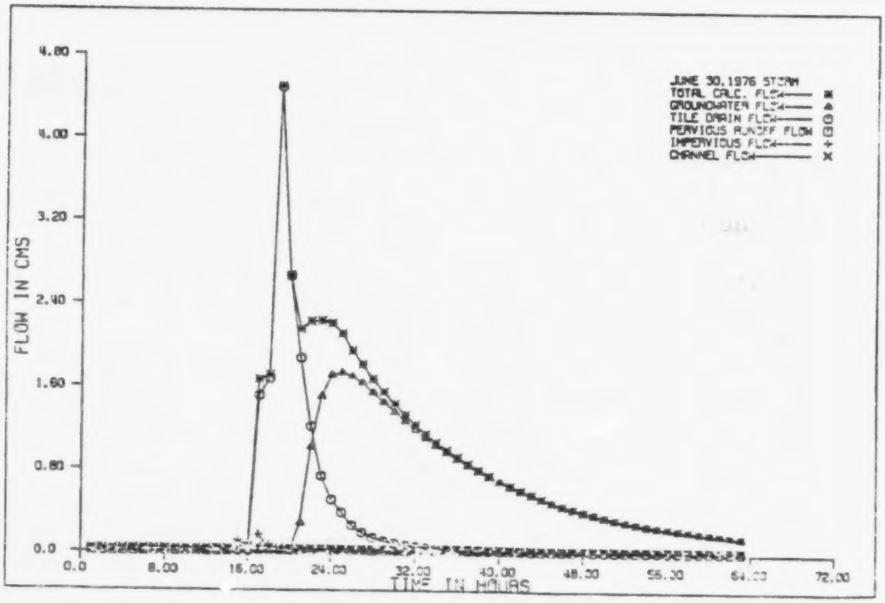


FIGURE 23 : JUNE 30 STORM - SUBHYD BREAKDOWN OF FLOW SOURCES

A farm by farm tile drain survey conducted as part of a PLUARG study in the West Canagagigue Creek catchment[19] determined that 40% of the catchment was systematically tile drained and that no strong correlation between soil type and tile drainage existed. Figure 8, introduced earlier is a composite of this survey.

Examination of this figure reveals that these agricultural tile drainage systems, including laterals, trunks and surface catch basins, are comparable in extent and complexity to urban storm drainage networks.

Conventional hydrologic models, including complex distributed ones such as those developed by Gupta (1974)[5] and Constable (1974)[1] have not explicitly considered the potential effects of tile drainage on runoff. Flood plain mapping studies in Ontario have also tended to neglect tile drains simply because there has been a lack of hydrologic models which include a representation of this mechanism. Hydrology textbooks also usually fail to mention or describe this runoff mechanism. The development of surface drains and tile systems incorporating catch basins, also represent an increase in drainage density, an underlying hydrologic parameter which has been shown to be very important.[6] Drainage basin development affects the fundamental hydrologic parameters of imperviousness and drainage density but conventional rural hydrologic models such as HYMO only allow for changes in imperviousness.

When possible impacts of tile drainage are discussed with farmers and agricultural engineers, they typically express the opinion that tiles do not increase runoff and may even decrease it and also that tiles do not flow during the summer.

However concern of individuals and the Ontario Ministry of the Environment regarding erosion at tile drain outfalls has reached the point where special outfalls designed to prevent erosion are being proposed. In this regard it would appear that thinking regarding development of rural drainage systems is parallel to but somewhat lagging that concerning urban drainage systems. At first urban storm sewers were developed with little concern for consequences downstream of the outfall. After some time erosion problems became evident and sewer outfalls were then designed to minimize erosion. Recently concerns regarding increased urban storm water flow and water quality have reached the point where attempts are now made to prevent increased runoff and adverse effects upon water quality.

Up to this time the potential impact of agricultural drainage systems regarding increased runoff has not been assessed. Due to this fact in Ontario governmental agencies have on the one had justifiably opposed the construction of urban drainage systems which could contribute to increased runoff; but on the other, encouraged and supported the development of more efficient agricultural drainage systems.

The Subcatchment Hydrologic Model was developed in such a way that a separate representation of each runoff mechanism considered significant was included. Tile drains were modelled by re-arranging a tile drain design methodology developed by Jan Van Schilfgaarde.[22] Van Schilfgaarde's method represents a slight improvement upon that of Glover which has been used by the U.S. Bureau of Reclamation and has been shown to produce reliable results.[23,24] Therefore it was decided to use SUBHYD to estimate what the catchment response to each of the three storms would have been if no tile drains had been present.

This was done by setting the percent tile drained area to zero but retaining the same initial moisture contents which had been previously determined to give the best simulation.

Figures 24 to 26 illustrate the SUBHYD simulations of these storm runoff events, both with and without tile drains.

The predicted reduction in peak flows from each storm were as follows:

- May 5-7, 1976 : 18%
- June 13, 1976 : 80%
- June 30, 1976 : 63%

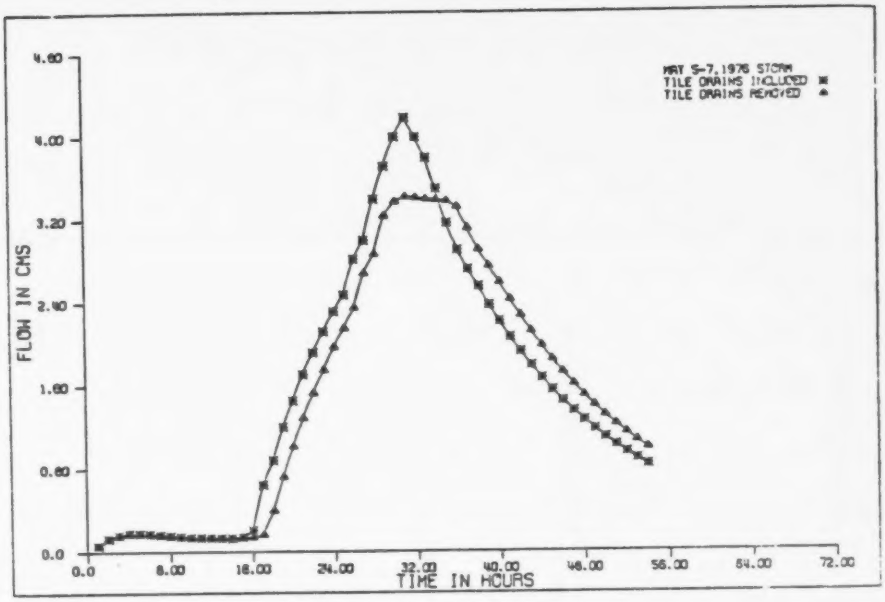


FIGURE 24 : MAY 5-7 STORM - EFFECT OF TILE DRAINS

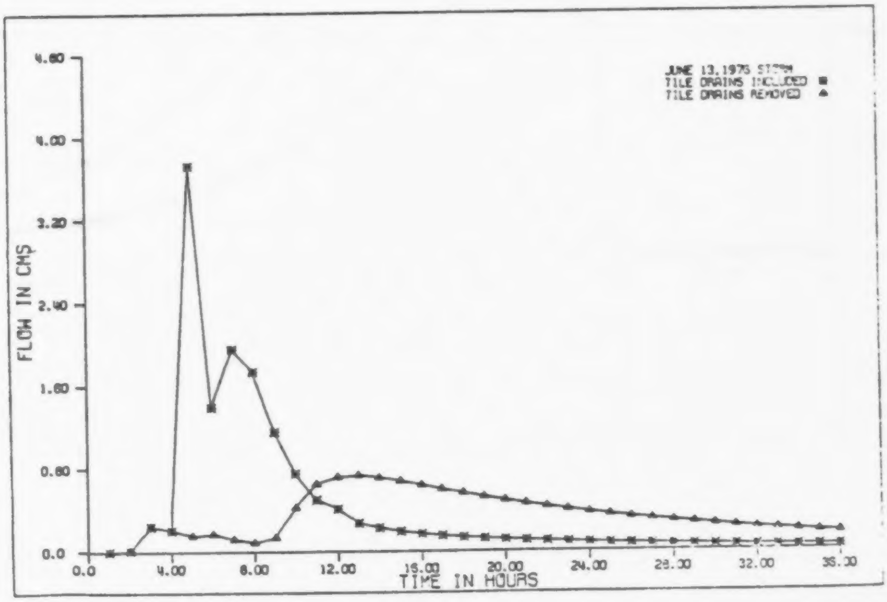


FIGURE 25 : JUNE 13 STORM - EFFECT OF TILE DRAINS

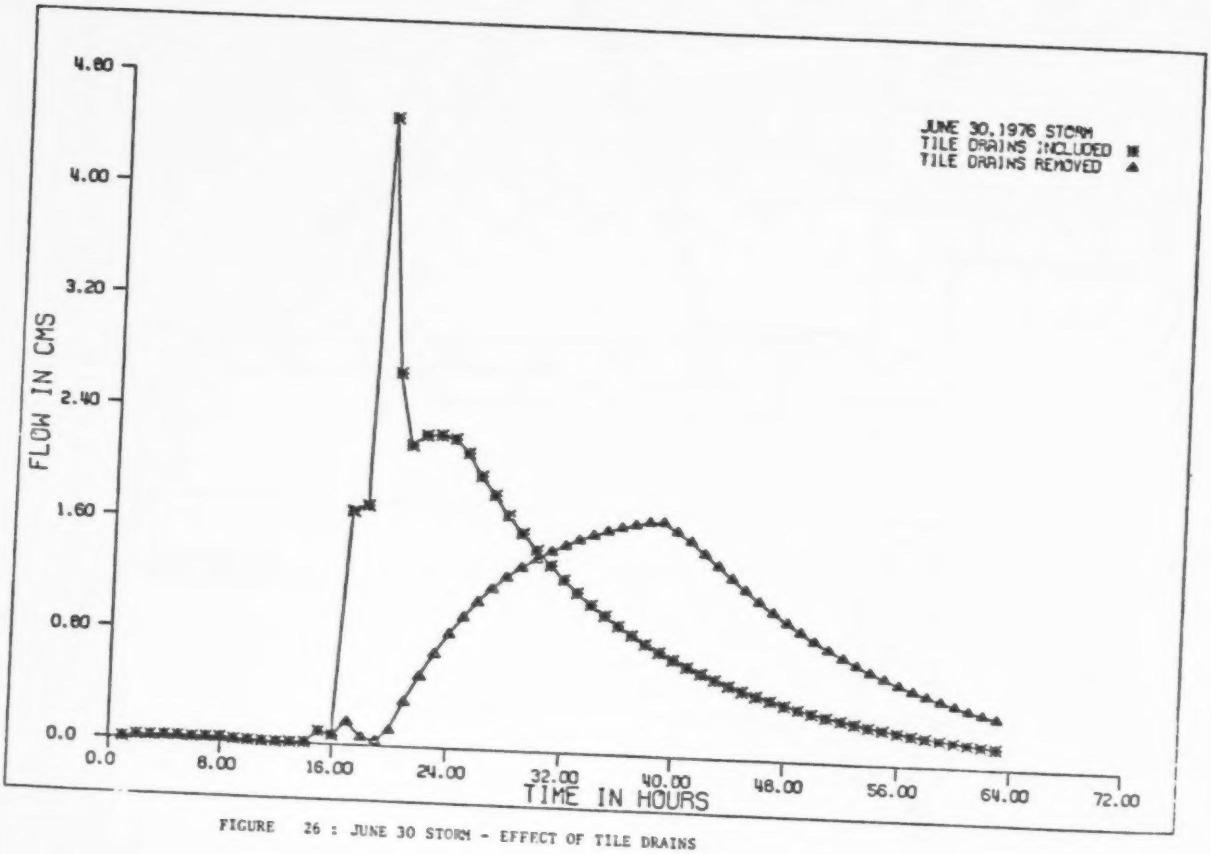


FIGURE 26 : JUNE 30 STORM - EFFECT OF TILE DRAINS

When the tiles were removed more water was able to percolate down to groundwater thereby increasing groundwater flow somewhat but still reducing total peak flows. The effect of removing tile drains was very similar to the damping effect that could be introduced by effective flood control storage in the system. A sharply peaking hydrograph resulting from a sudden, intense storm was very sensitive to the removal of tile drains, whereas a gradual damped hydrograph was not so sensitive. The physical explanation of this phenomenum is as follows:

An intense storm produces higher rates of infiltration by approaching or exceeding infiltration capacity, and consequently creates greater hydraulic heads on the tile drains. This greater head then causes tiles to flow at greater rates than for a less intense storm. This is borne out by the SUBHYD final results and becomes especially evident when the respective heads on the tile drains are monitored in SUBHYD.

SUMMARY

A subcatchment hydrologic model (SUBHYD) has been developed and implemented on a micro computer. This model is a collection of mathematical representations of various components in the hydrologic cycle and as such should be updated as better techniques of modelling these components become available.

The model has been tested on an urban subcatchment of 18.2 hectares (45 acres) and a rural subcatchment of 19.46 sq. km. (4,800 acres).

By varying only initial soil moisture content, the model was able to simulate observed runoff within an acceptable degree of accuracy, from four storms ranging in size from 1.96 cm (0.77 in.) to 5.2 cm (2.05 in.). The model's performance for these storms compared favourably with that of two other hydrologic models.

The model indicates that under intense storm conditions the presence of substantial agricultural tile drains significantly affects runoff hydrographs, tending to increase peak flows.

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RAINFALL-RUNOFF MODELING OF FLOW AND TOTAL NITROGEN FROM TWO LOCALITIES IN THE DENVER, COLORADO, METROPOLITAN AREA

By

William M. Alley and Sherman R. Ellis

U.S. Geological Survey

Presented at the

Storm Water Management Model Users

Group Meeting

Annapolis, Maryland

November 13-14, 1978

RAINFALL-RUNOFF MODELING OF FLOW AND TOTAL NITROGEN FROM TWO LOCALITIES IN THE DENVER, COLORADO, METROPOLITAN AREA

By William M. Alley[1] and Sherman R. Ellis[2]

ABSTRACT

Data from two residential basins in the Denver, Colorado, metropolitan area were used for calibration and verification of the U.S. Environmental Protection Agency's Storm Water Management Model II for rainfall-runoff modeling of flow and total nitrogen. Using values for the accumulation and washoff parameters as suggested in the user's manual and basin-characteristic parameters as determined from the flow calibration, the errors in simulated storm-runoff loads of total nitrogen ranged from 67 to 1,000 percent. The model assumption that land-surface loads are directly proportional to the number of days prior to the storm during which the accumulated rainfall was less than 1.0 inch was not substantiated. Results indicate that the model simulations of total nitrogen are very sensitive to the poorly defined model parameters specifying the part of the suspended-solids

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[1] Hydrologist, U.S. Geological Survey, Reston, Virginia

[2] Hydrologist, U.S. Geological Survey, Lakewood, Colorado

and settleable-solids concentrations added to the total nitrogen concentration. Based on the results of this study, caution should be used when applying the Storm Water Management Model II or other similar models to assist in making water-quality-management decisions, particularly in the absence of local data for model calibration and verification.

INTRODUCTION

The effects of urban runoff on water quality are of concern in most metropolitan areas in the United States. This concern has been largely a result of the areawide planning requirements of Section 208 of Public Law 92-500, 1972 Amendments to the Federal Water Pollution Control Act.

Concomitant with this rising concern about urban-runoff quality has been the development of numerous computer models for simulating the storm-runoff and water-quality processes. Unfortunately, development of these models is based on a very small data set. It has been suggested that development of these models has " * * * seemingly already greatly outpaced the data base for model verification * * * " (McPherson, 1975). Additional data appear to be required before substantial improvements in storm-runoff-quality modeling in urban areas can be achieved (Whipple, 1974).

The purpose of this paper is to present the results of calibration and verification of the U.S. Environmental Protection Agency's Storm Water Management Model II (Huber and others, 1975) for simulating total nitrogen concentrations and loads in rainfall runoff. Modeling of urban-runoff quality requires conjunctive consideration of the quantity and quality aspects of urban runoff (Alley, 1976). Therefore, the results of rainfall-runoff modeling of flow are also presented. The data used in this study were collected by the U.S. Geological Survey during 1976 and 1977 as part of separate but coordinated cooperative agreements with several local governmental entities--the Denver Board of Water Commissioners, the Denver Regional Council of Governments, and the Urban Drainage and Flood Control District. Data collected during the investigation have been published by Ellis (1978).

DESCRIPTIONS OF DRAINAGE BASINS

Data from two drainage basins were utilized in this modeling study (fig. 1). One drainage basin is located in Littleton, Colo. (the Littleton site) and has an area of 606 acres. Approximately 75 percent of the basin contains single-family dwellings and 25 percent is parks and open space. About 15 percent of the basin is covered by effective impervious surfaces[1] and 10 percent by noneffective impervious surfaces[2]. Street gutters and open ditches are the predominant drainage types for flow conveyance.

A second drainage basin is located in Lakewood, Colo. (the Lakewood site) and has an area of 76.7 acres. Approximately 50 percent of the basin is undeveloped land, 30 percent is multifamily dwellings, and 20 percent is commercial development. About 30 percent of the basin is covered by effective impervious surfaces and 10 percent by noneffective impervious surfaces. Street gutters convey storm runoff to a storm sewer crossing the basin.

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[1] Effective impervious areas are impervious areas which are connected and, in turn, connect to some means of conveying the runoff out of the areas, such as roofs which drain onto driveways, streets, sidewalks, and paved parking lots.

[2] Noneffective impervious areas are impervious areas which are not connected to other impervious areas and which drain to pervious areas, such as roofs which drain onto lawns.

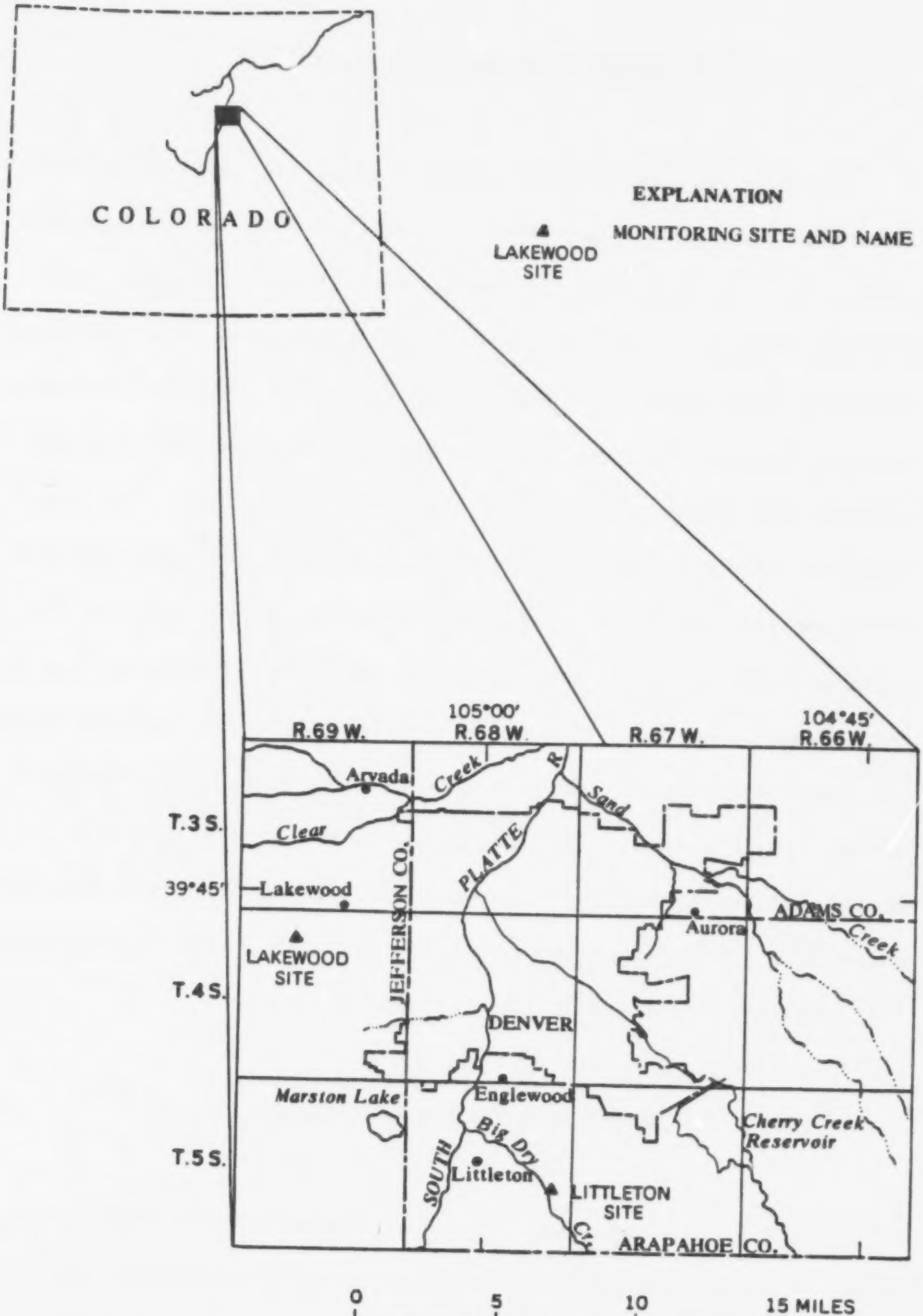

COLORADO
EXPLANATION
LAKEWOOD SITE
MONITORING SITE AND NAME
105°00′
104°45′
R.69 W.
R.68 W.
R.67 W.
R.66 W.
T.3 S.
T.4 S.
T.5 S.
39°45′
Arvada
Clear
Creek
R
Sand
PLATTE
Lakewood
JEFFERSON CO.
ADAMS CO.
Aurora
Creek
LAKEWOOD SITE
DENVER
Marston Lake
Englewood
Big Dry
SOUTH
Littleton
LITTLETON SITE
Cr.
Cherry Creek Reservoir
ARAPAHOE CO.
0
5
10
15 MILES
0
5
10
15 KILOMETERS

Figure 1.--Locations of urban-runoff monitoring sites.

INSTRUMENTATION AND DATA COLLECTION

The data utilized in this study were collected using automatic water-quality-sampling and rainfall-runoff monitors. The automatic monitor and its operation are described by Smoot, Davidian, and Billings (1974). Flows were determined and samples were collected at the outlets of the drainage basins using the automated equipment. The gage height-discharge relationship for the Lakewood site was derived from theoretical ratings using a V-notch weir. The gage height-discharge relationship for the Littleton site was computed using the step-backwater approach (Shearman, 1976). Samples for water-quality analysis were collected at intervals ranging from 2 to 5 minutes depending upon basin size and experience gained during the sampling program. These intervals were occasionally changed during runoff periods. All samples were analyzed at the U.S. Geological Survey regional laboratories. Three tipping-bucket rain gages were installed in the Littleton basin, and one tipping-bucket rain gage was installed in the Lakewood basin.

MODEL APPLICATION AND RESULTS

The U.S. Environmental Protection Agency's Storm Water Management Model II (Huber and others, 1975) is a computer model for the simulation of storm and combined sewerage. Application of the model requires that the basin be divided into "homogeneous" subbasins.

Overland flow from these subbasins is routed to the basin outlet through a gutter-and-pipe network. Subbasins and the gutter-and-pipe network are delineated for the Littleton and Lakewood basins in figures 2 and 3.

Flow

During this study, six periods of rainfall runoff were gaged at each site for a sufficient duration and at a sufficient frequency so that storm-runoff volumes and peak flows could be determined accurately. Three of these periods of rainfall runoff at each site were selected for model calibration and three at each site for model verification. Runoff periods selected for flow calibration were those to be used for total nitrogen calibration.

Prior to calibrating the model, a literature review was used to obtain an improved understanding of model-output sensitivity to changes in the input parameters. Three studies have included sensitivity analysis of the model (Graham and others, 1974; Huber and others, 1975; and Proctor and Redfern Limited and James F. MacLaren Limited, 1976). During each of these studies, all model parameters except one were held constant. The model parameter being tested was then varied through a range of values greater and lesser than the accepted mean value. All studies concluded that model output was most sensitive to the amount of effective impervious

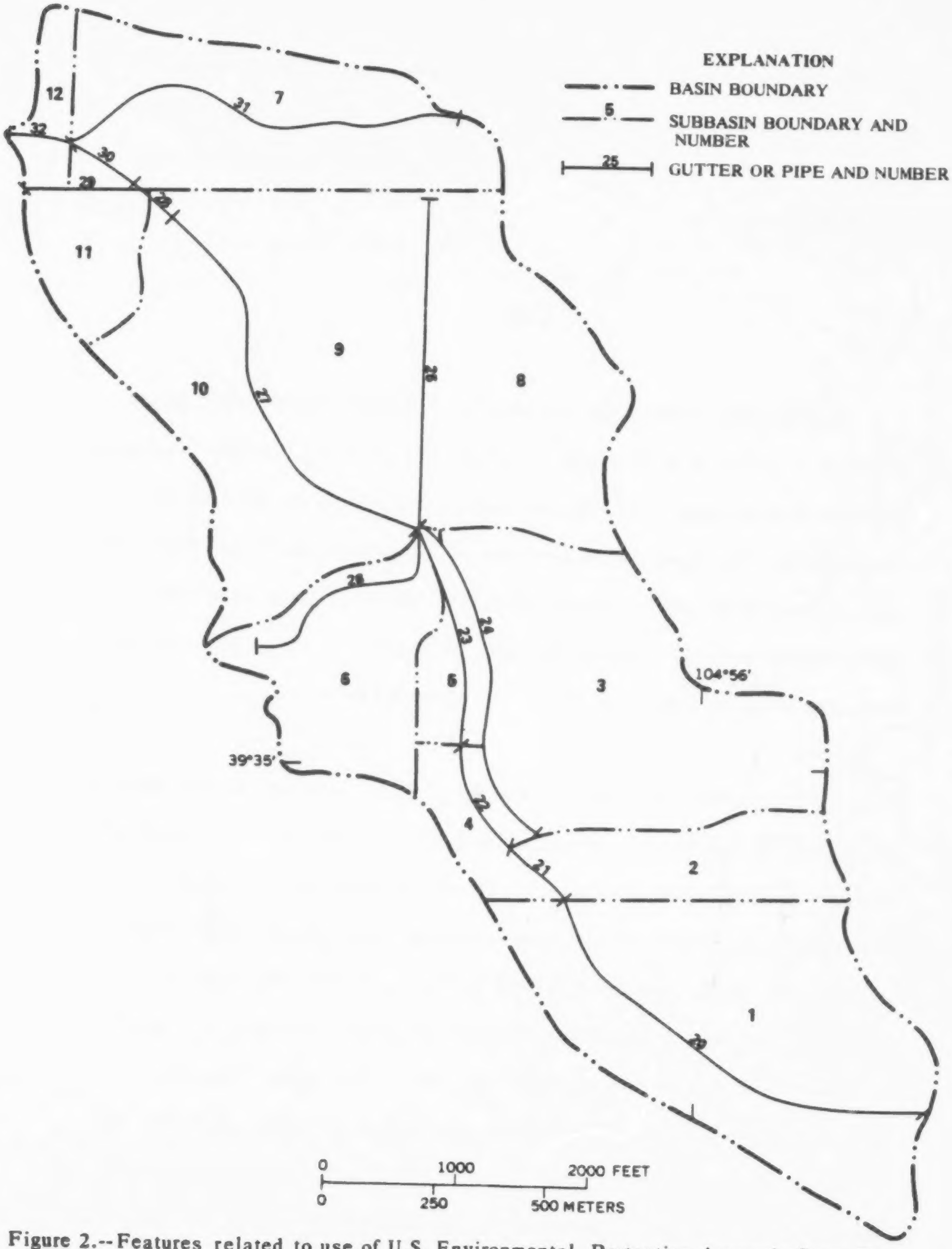
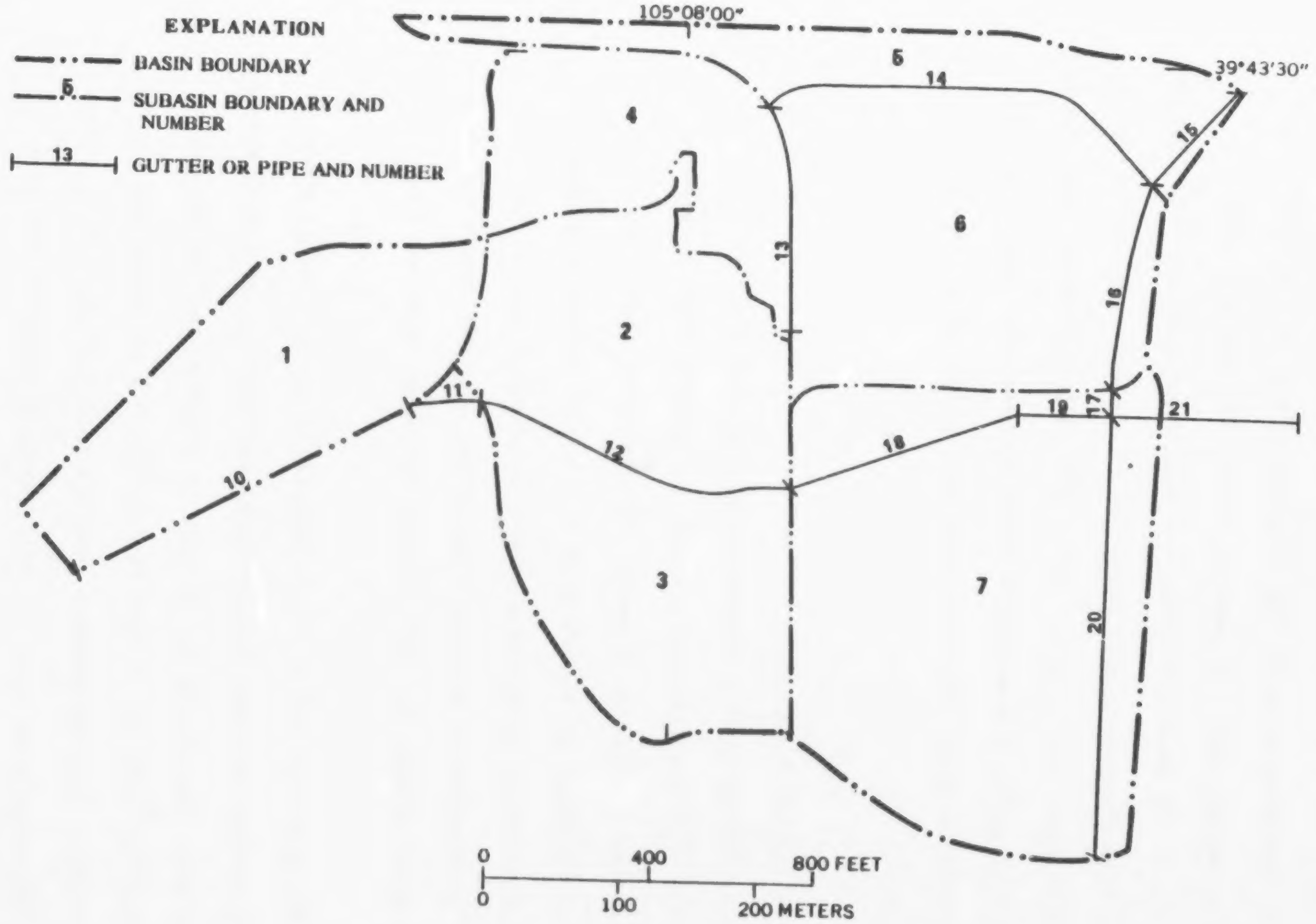

EXPLANATION
BASIN BOUNDARY
5
SUBBASIN BOUNDARY AND NUMBER
25
GUTTER OR PIPE AND NUMBER
12
7
31
32
30
29
11
9
10
27
26
8
28
24
23
6
5
3
104°56'
39°35'
22
4
21
2
1
20
0
1000
2000 FEET
0
250
500 METERS

Figure 2.--Features related to use of U.S. Environmental Protection Agency's Storm Water Management Model II for the Littleton basin.


EXPLANATION
BASIN BOUNDARY
5
SUBASIN BOUNDARY AND NUMBER
13
GUTTER OR PIPE AND NUMBER
105°08'00"
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11
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13
14
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17
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19
20
21
0
400
800 FEET
0
100
200 METERS

Figure 3.--Features related to use of U.S. Environmental Protection Agency's Storm Water Management Model II for the Lakewood basin.

area. Sensitivity of the other model parameters was somewhat variable depending on basin characteristics and the range of parameter values used. In general, runoff volumes were most sensitive to the amount of effective impervious area and infiltration rates. Peak flows were most sensitive to the amount of effective impervious area, infiltration rates, and subbasin widths. Additional sensitivity analysis made by the authors showed both runoff volumes and peak flows to be sensitive to assumed errors of ±5 percent in basin area.

The initial simulations for model calibration were made using the basin-characteristic parameters as reported by Ellis (1978) and values for the infiltration and retention-storage parameters as suggested in the user's manual for the model (Huber and others, 1975). The model parameters were then varied during trial-and-error calibration guided by a knowledge of model sensitivity, experience gained in previous simulations, and the ranges within which model parameters realistically may be adjusted.

The model for all initial simulations underestimated storm-runoff volumes and peak flows. Model calibration was then performed as follows: During the first stage of calibration, the maximum infiltration rate was changed from 3.0 to 0.5 in./h and the minimum infiltration rate was changed from 0.5 to 0.2 in./h. These values were more consistent with the soil characteristics of the basins

(Larsen and Brown, 1971). Effective impervious areas for each subbasin were then multiplied by a factor as much as 1.5. This accounted for the effect of errors in determining effective impervious areas, as well as the effect of noneffective impervious areas. If the average ratio of simulated runoff volume to measured runoff volume could not attain 1.0 using this strategy, then all subbasin areas were multiplied by a factor as much as 1.05 until this ratio was 1.0. A "best" fit was then obtained in the timing of hydrograph peaks and troughs by multiplying all subbasin widths by a factor as much as 1.25. Upon completion of this step, the impervious-area multiplication factor was reduced, if necessary, to achieve an average ratio of simulated runoff volume to measured runoff volume of 1.0. The model was then considered calibrated.

Littleton Site

Runoff periods of May 30, 1976, July 25, 1976, and June 11, 1977, were selected for model calibration. Runoff volumes for initial (uncalibrated) simulations averaged 63 percent of the measured runoff volumes and the peak flows averaged 42 percent of the measured peak flows.

Model calibration was then performed. The final calibrated model had maximum and minimum infiltration rates of 0.5 and 0.2 in./h, respectively. In addition, all subbasin areas were multiplied by

1.05, impervious areas for each subbasin were multiplied by 1.48, and all subbasin widths were multiplied by 1.25. The results for the calibrated model are shown in figure 4. The model simulated peak flows which averaged 79 percent of those measured.

Verification simulations were then made for runoff periods of September 7, 1976, April 15, 1977, and April 19, 1977, using the model parameters determined during model calibration. The results of model verification are shown in table 1 and in figure 5. The timing of hydrograph peaks and troughs was similar for the measured and simulated hydrographs. However, the simulated runoff volumes and peak flows were significantly larger than the measured runoff volumes and peak flows for the runoff periods of September 7, 1976, and April 19, 1977. The measured and simulated hydrographs of April 15, 1977, were similar.

Lakewood Site

Runoff periods of April 12, 1977, April 19-20, 1977, and May 20, 1977, were selected for model calibration. Runoff volumes for initial (uncalibrated) simulations averaged 86 percent of the measured runoff volumes for April 12, 1977, and April 19-20, 1977. The simulated runoff volume for May 20, 1977, was about 40 percent of the measured runoff volume. The runoff of May 20, 1977, which

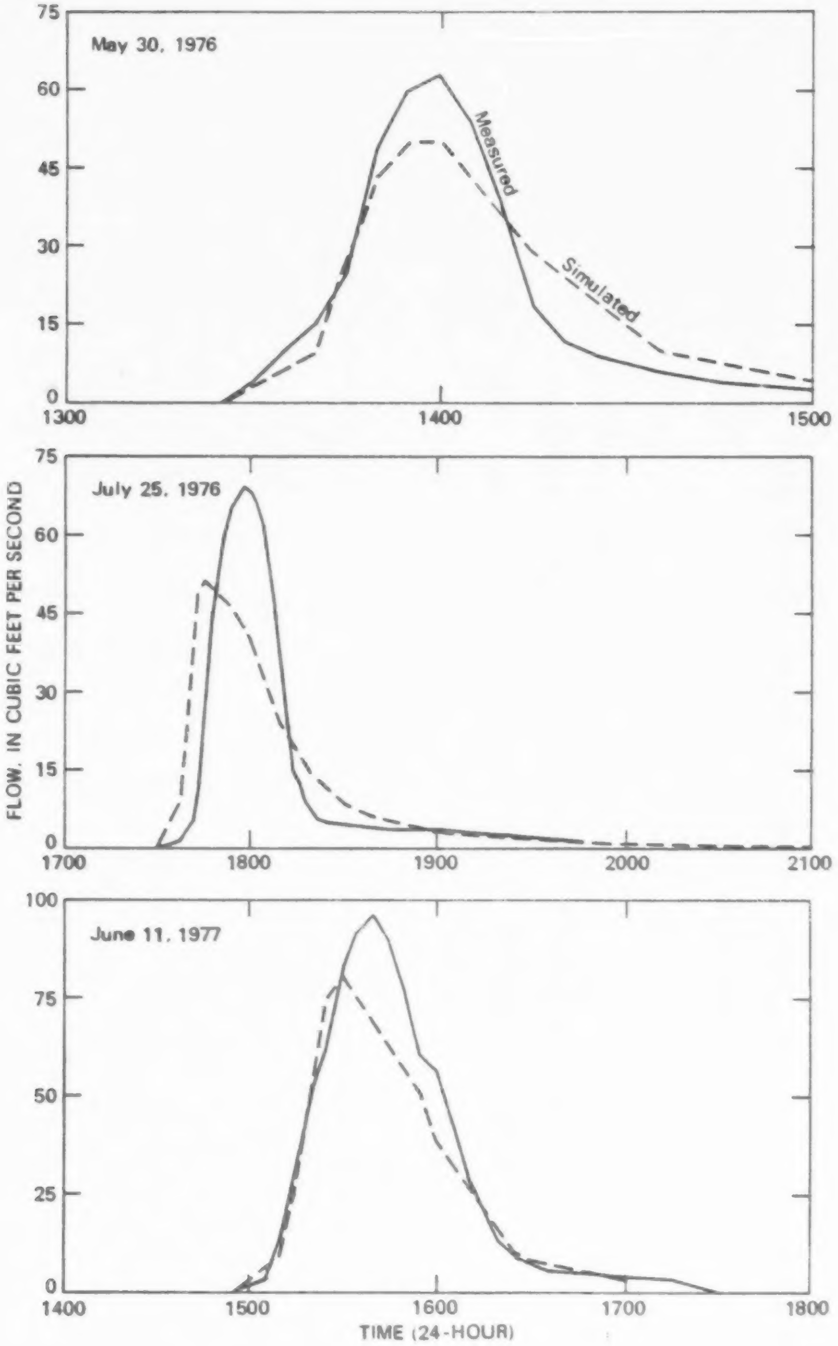

Figure 4.--Simulated and measured hydrographs for runoff periods used in model calibration for the Littleton site.

Table 1.--*Measured and simulated storm-runoff volumes and peak flows at the Littleton site*

Date	Measured runoff volume (in.)	Simulated runoff volume (in.)	Percent error	Measured peak flow (ft^3/s)	Simulated peak flow (ft^3/s)	Percent error
Sept. 7, 1976-----	0.07	0.14	100	52	77	48
April 15, 1977----	.17	.20	18	33	30	-9
April 19, 1977----	.08	.13	62	14	18	29

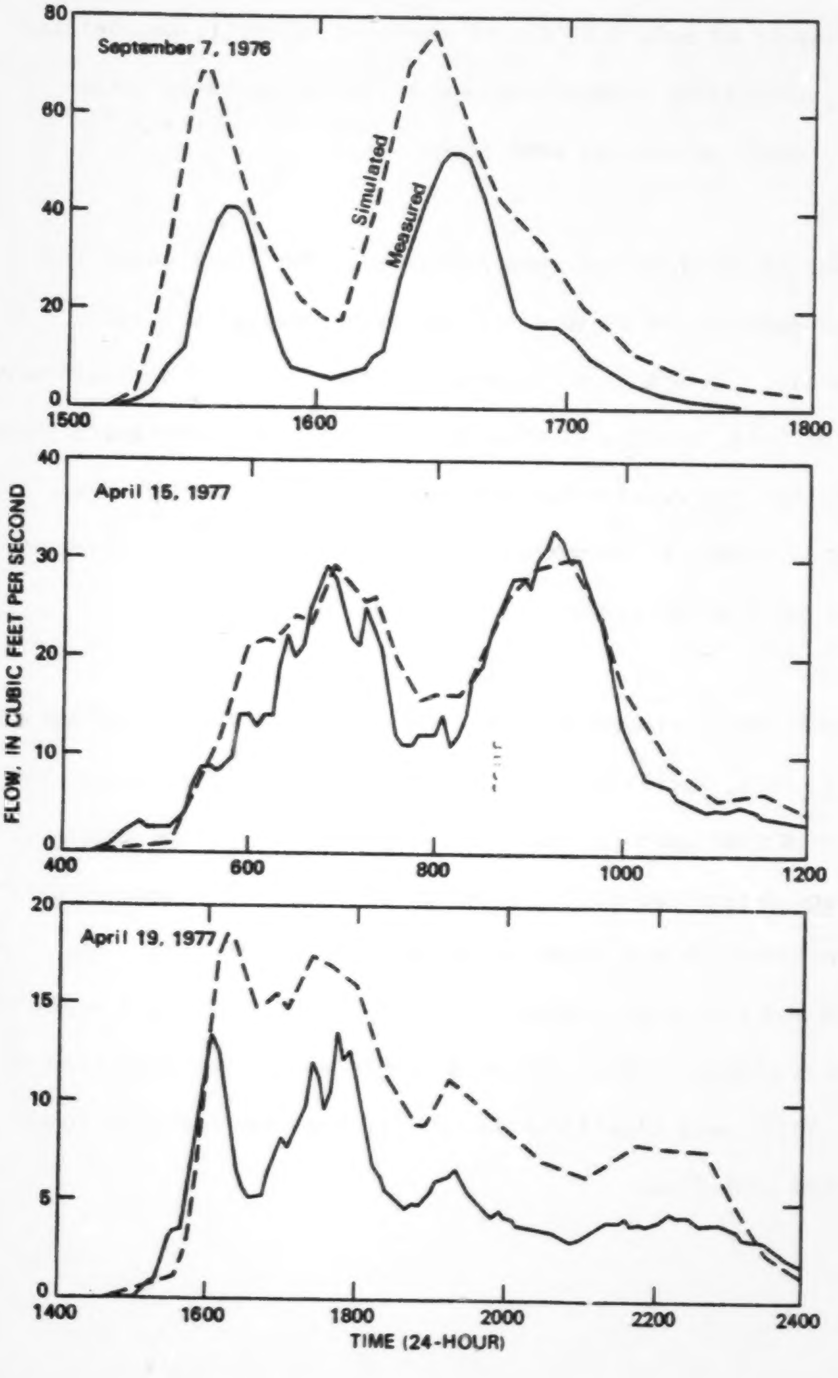

Figure 5.—Simulated and measured hydrographs for runoff periods used in model verification for the Littleton site.

was the result of only 0.07 in. of measured rainfall, was deleted from the calibration process because of its anomalously large measured runoff volume and peak flow.

Model calibration was then performed. The final calibrated model had maximum and minimum infiltration rates of 0.5 and 0.2 in./h, respectively. In addition, impervious areas for each subbasin were multiplied by 1.10. The results for the calibrated model are shown in figure 6. The results for the runoff of May 20, 1977, are included in figure 6, even though this runoff period was eliminated from the calibration process.

Verification simulations were then made for runoff periods of April 11, 1977, April 15, 1977, and May 20, 1977 (a different runoff period than that used for model calibration), using the model parameters determined during model calibration. The results of model verification are shown in table 2 and in figure 7. The measured and simulated hydrographs of April 11, 1977, and April 15, 1977, were similar. The simulated runoff volume and peak flow of May 20, 1977, were significantly smaller than the measured runoff volume and peak flow.

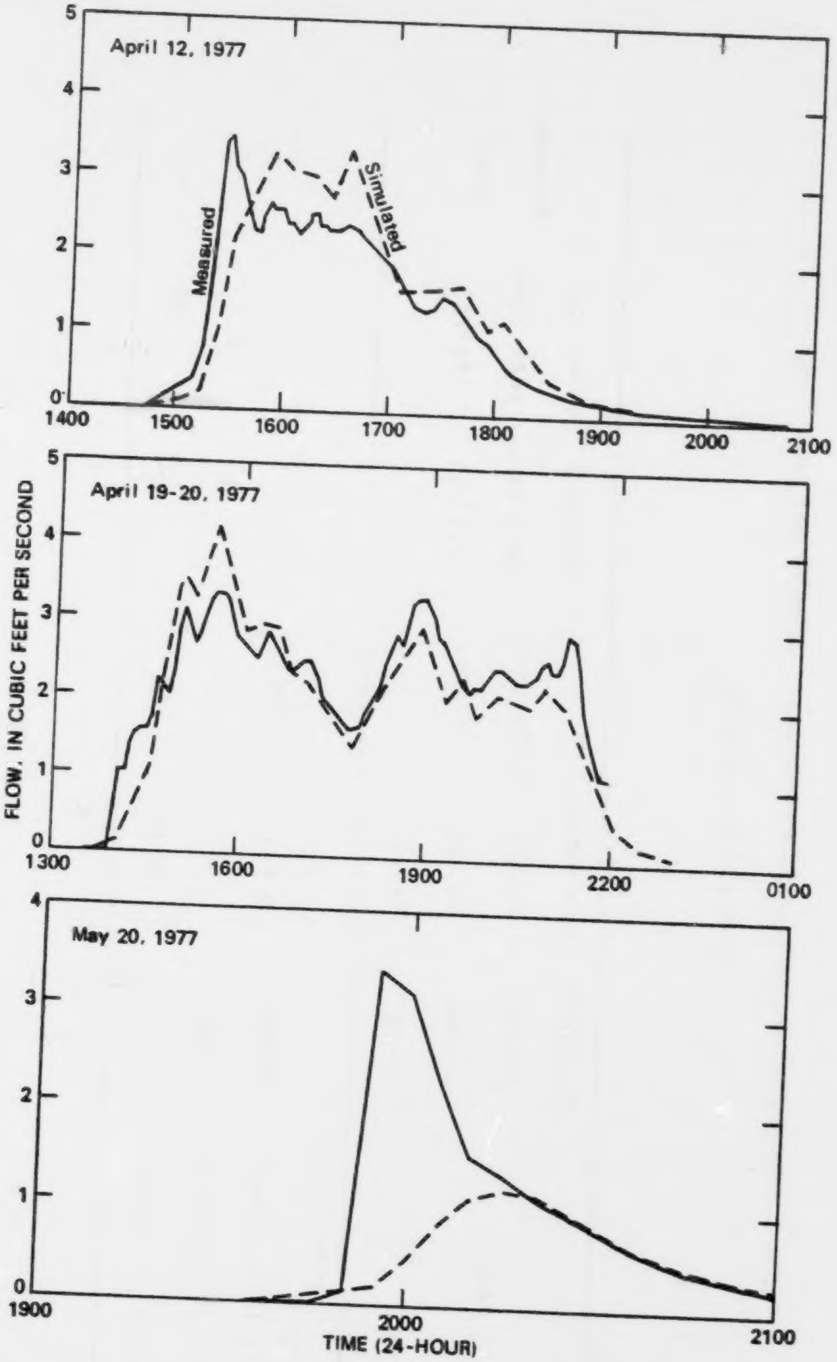

Figure 6.—Simulated and measured hydrographs for runoff periods used in model calibration for the Lakewood site.

Table 2.--*Measured and simulated storm-runoff volumes and peak flows at the Lakewood site*

Date	Measured runoff volume (in.)	Simulated runoff volume (in.)	Percent error	Measured peak flow (ft^3/s)	Simulated peak flow (ft^3/s)	Percent error
1977						
April 11----------	0.036	0.043	19	3.4	3.6	6
April 15----------	.16	.16	0	4.3	4.7	9
May 20------------	.0092	.0039	-58	1.3	.38	-71

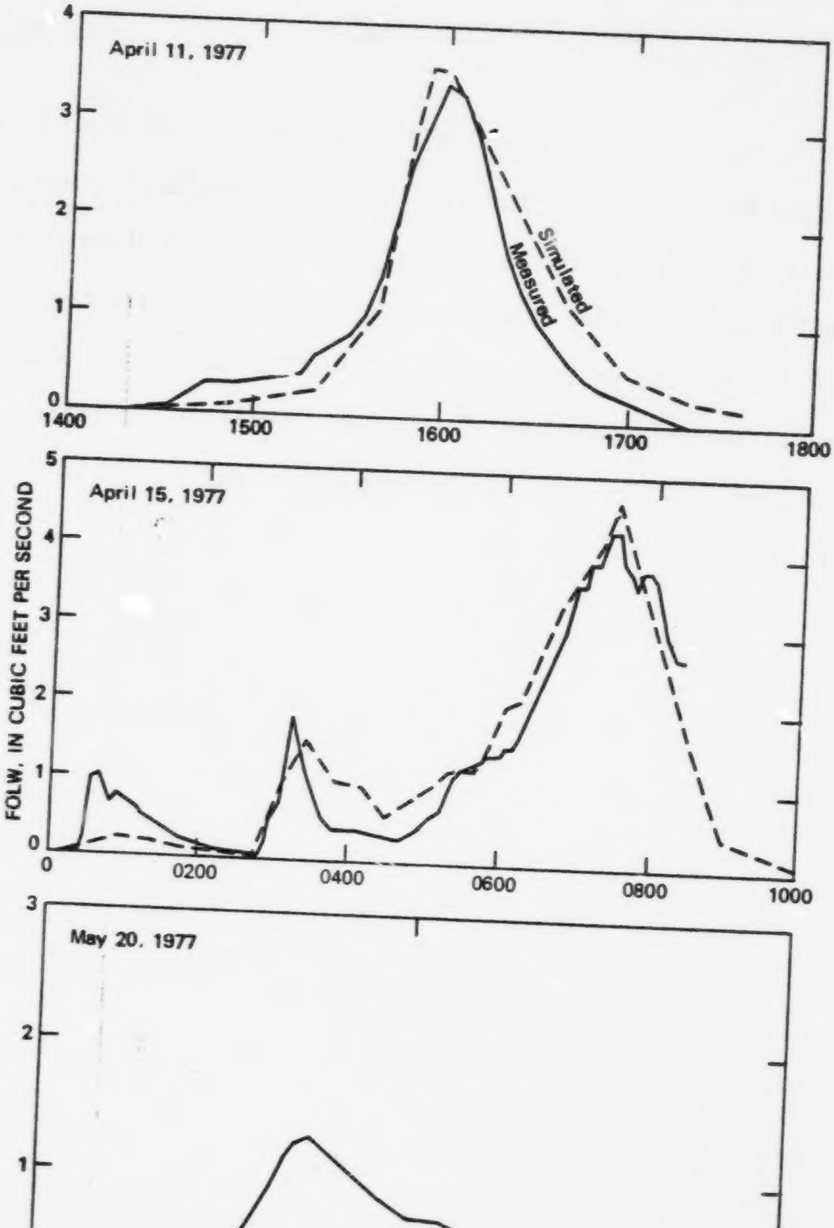

Figure 7.--Simulated and measured hydrographs for runoff periods used in model verification for the Lakewood site.

Total Nitrogen

The mathematical algorithms for rainfall-runoff quality in the U.S. Environmental Protection Agency's Storm Water Management Model II are the relationships most widely used in urban storm-runoff-quality models. The model will simulate settleable solids, suspended solids, coliform bacteria, biochemical oxygen demand, chemical oxygen demand, total nitrogen, and total phosphate. Of these constituents, only total nitrogen was determined in samples of rainfall runoff.

There are two main components of the rainfall-runoff-quality algorithms in the model: Constituent accumulation and constituent washoff. Constituent accumulation is estimated as a function of land use, the number of days prior to the storm during which the accumulated rainfall is less than 1.0 in., and street-cleaning frequency and efficiency. The constituent load on the ground surface (land-surface load) at the start of a storm is modeled as:

$$LLOAD = QFACT \times DFACT \times DRYDAYS \times GLEN \times 453.6, \qquad (1)$$

where

LLOAD=land-surface load, in milligrams;

QFACT=milligram of constituent per gram of "dust and dirt";

DFACT="dust and dirt" loading rate for each land use, in pounds per day per foot of curb;

DRYDAYS=number of days prior to the storm during which the accumulated rainfall is less than 1.0 in. (modified to account for street sweeping); and

GLEN=curb length, in feet.

453.6=conversion factor (grams per pound).

The parameter values suggested in the user's manual for the model (Huber and others, 1975) for total nitrogen accumulation are shown in table 3.

Table 3.--*Recommended values for total nitrogen accumulation parameters*

[From Huber and others, 1975]

Land use	DFACT "Dust and dirt" accumulation (pound per day per 100 feet of curb)	QFACT Milligrams of total nitrogen as N per gram of "dust and dirt"
Single-family residential---	0.7	0.48
Multifamily residential-----	2.3	.61
Commercial------------------	3.3	.41
Industrial------------------	4.6	.43
Parks and open space--------	1.5	.05

Washoff of constituents is modeled using an exponential decay equation:

$$POFF = PO[1-e^{(-BxRx\Delta t)}], \qquad (2)$$

where

POFF=amount of constituent removed from land surface during a time step, in milligrams;

PO=amount of constituent on land surface at beginning of time step, in milligrams;

B=decay coefficient;

R=runoff rate, in inches per hour; and

Δt=time step, in hours.

The primary assumption for use of this equation is that the amount of constituent washed off during each time step is proportional to the amount remaining on the land surface at the beginning of the time step. The rate of washoff is controlled by the runoff rate (R) for each time step and the exponent B. The exponent B is set at a constant value of 4.6, assuming that a uniform rainfall of 0.5 in./h would wash off 90 percent of the constituent in 1 hour. For total nitrogen, the model assumes that 4.5 percent of the suspended-solids concentration and 1.0 percent of the settleable-solids concentration simulated by the model should be added to the nitrogen concentration computed from equation 2.

Littleton Site

The same runoff periods used in the flow calibration, May 30, 1976, July 25, 1976, and June 11, 1977, were used for the total nitrogen calibration. The initial simulations were made using values for the accumulation and washoff parameters as suggested in the user's manual and values for the infiltration and basin-characteristic parameters as determined from the flow calibration.

Comparisons of simulated with measured storm-runoff loads of total nitrogen for the initial simulations are shown in table 4. Simulation errors ranged from 140 to 1,000 percent. The extent of this overprediction increases with the value of the model parameter, DRYDAYS (number of days prior to a storm during which the accumulated rainfall was less than 1.0 in.). This is consistent with the fact that the data from the Littleton basin show no discernible effect of DRYDAYS on storm-runoff loads of total nitrogen (fig. 8); whereas, the model assumes that the land-surface load of total nitrogen at the start of a storm is directly proportional to DRYDAYS.

Table 4.--*Measured and simulated storm-runoff loads of total nitrogen from the Littleton basin*

Date	DRYDAYS[1]	Measured load (pounds)	Simulation 1[2]		Simulation 2[3]	
			Simulated load (pounds)	Percent error	Simulated load (pounds)	Percent error
May 30, 1976---	31	30	72	140	3.1	-90
July 25, 1976--	56	30	240	700	7.3	-76
June 11, 1977--	57	38	420	1,000	13	-66

[1] DRYDAYS is a model parameter set at the number of days prior to the storm during which the accumulated rainfall is less than 1.0 in.

[2] Model constituent-accumulation and constituent-washoff parameters were set at those values suggested by model documentation (Huber and others, 1975).

[3] Same as simulation 1 except no part of the suspended-solids or settleable-solids concentration was added to the total nitrogen concentration.

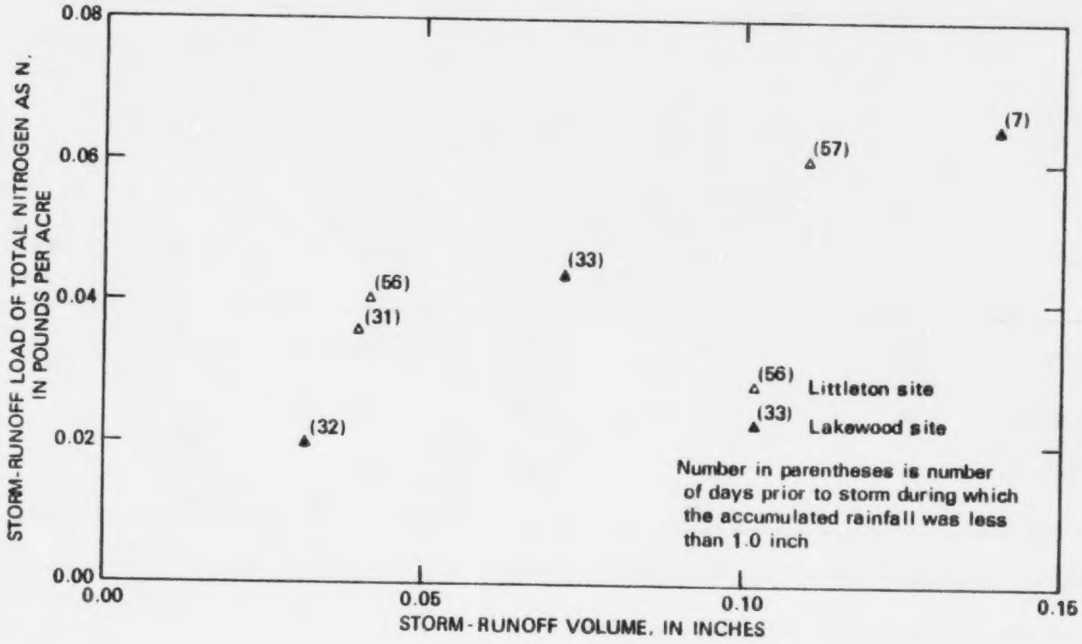

Figure 8.--Relation between storm-runoff load of total nitrogen and storm-runoff volume.

A similar lack of discernible effects of number of days since the preceding rainfall on storm-runoff loads has been reported by Whipple and others (1977). Their results have been challenged by Field (1978) and Lager (1978) because the storms used in their analysis were of such small total storm volumes that very little of the land-surface loads was washed off. However, for the three runoff periods of table 4, the percentage of the land-surface load washed off (according to the model) varied from 28 to 48 percent.

A second simulation was made assuming that no part of the suspended-solids or settleable-solids concentration was added to the total nitrogen concentration. The results, shown in table 4, indicate that the model simulations of total nitrogen are very sensitive to these poorly defined parameters.

Most computer models which simulate the quality of rainfall runoff from urban areas contain similar algorithms to those used in the Storm Water Management Model II. Likewise, many of these models have a similar set of recommended values for the model parameters used in the runoff-quality algorithms. These models have been widely used as a source of input for making management decisions. The large simulation errors obtained for the Littleton site suggest that caution should be used when applying these models to assist in making water-quality-management decisions, particularly in the absence of local data for model calibration and verification.

A model option to specify the land-surface load for each subbasin, in pounds per acre, at the beginning of storm runoff was then used in place of the constituent-accumulation equation. The land-surface loads for each of the three runoff periods were adjusted until the ratio of simulated to measured storm-runoff load was 1.0. Because of the lack of any other data, the land-surface loads were allocated between land uses in the same proportions as the values of DFACT x QFACT from table 3. The results of the "calibrated" model for each runoff period are shown in table 5. No relationship of basin-average land-surface load with number of days since last street sweeping is displayed by these results. In addition, the rainfall that occurred before the runoff of May 30, 1976 (0.17 in. on May 20 and 0.28 in. on May 21), did not appear to have noticeably affected the land-surface load at the beginning of storm runoff on May 30.

The results of the three calibration simulations are shown in figure 9. These indicate that the simulated and measured rates of decreasing total nitrogen concentrations with elapsed time from the start of a storm are similar, resulting in some confidence in the constituent-washoff equation (equation 2).

Three additional runoff periods were sampled for total nitrogen but an insufficient number of samples were collected to provide meaningful storm-runoff loads. However, these runoff periods could be used for model verification of total nitrogen concentrations.

Table 5.--*Basin-average land-surface loads of total nitrogen at the beginning of storm runoff based on model calibration for the Littleton basin*

Date	Daily rainfall for previous 10 days[1] (inches)										Number of days since last street sweeping	Basin-average land-surface load of total nitrogen as N at the beginning of storm runoff (pound per acre)
	10	9	8	7	6	5	4	3	2	1		
May 30, 1976---	0.17	0.28	0	0	0.02	0	0	0	0	0	10	0.16
July 25, 1976--	0	0	0	0	.06	0	0	0	0	0	66	.17
June 11, 1977--	0	0	0	0	0	0	0	0	0	0	>10	.12

[1] Refers to number of days prior to date of storm runoff.

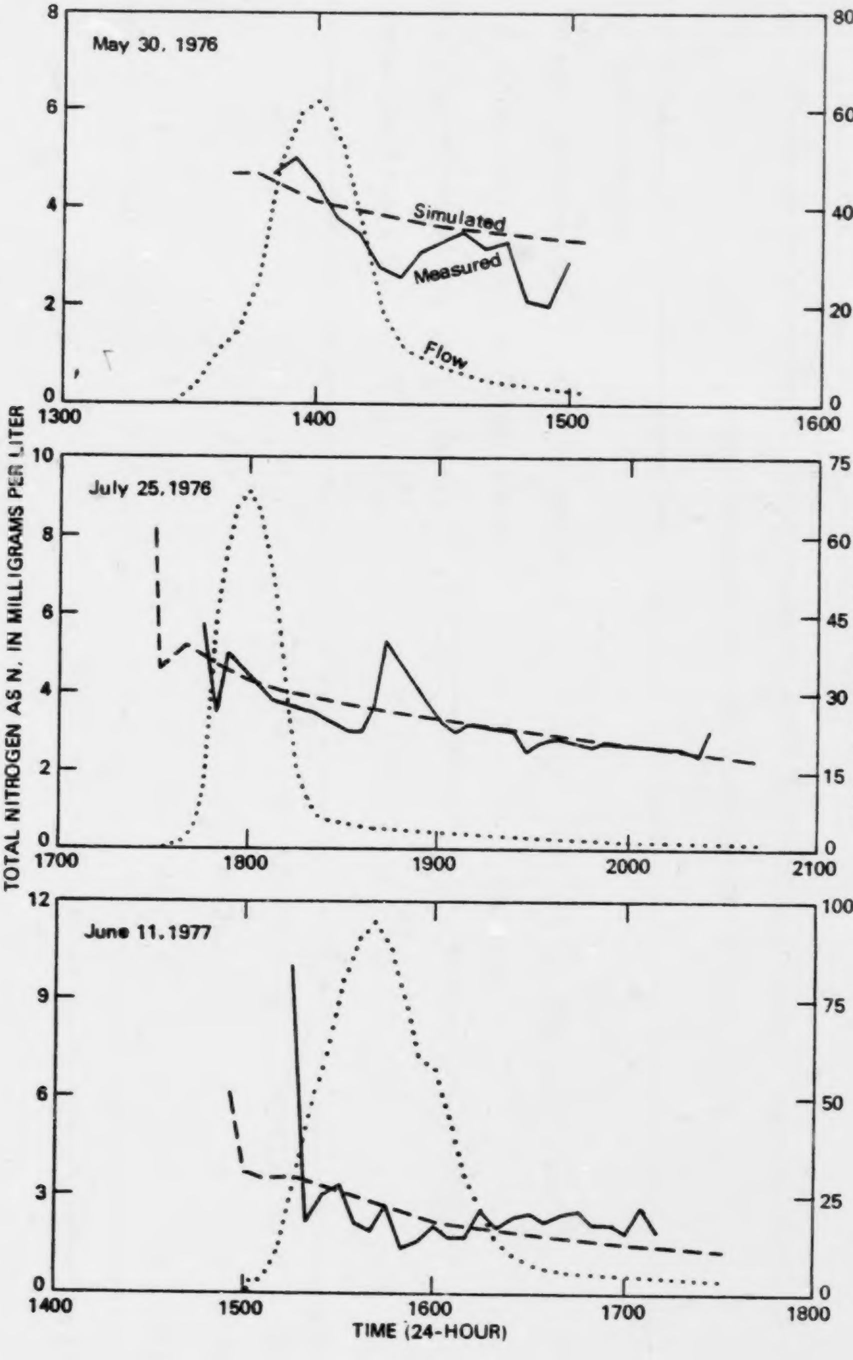

Figure 9.--Simulated and measured changes in total nitrogen with time for runoff periods used in model calibration for the Littleton site.

Basin-average land-surface load of total nitrogen was assumed to be equal to the average of the basin-average land-surface loads shown in table 5. The results of the three verification simulations are shown in figure 10.

Lakewood Site

The runoff periods of July 19, 1976, April 12, 1977, and April 19-20, 1977, were used for model calibration. Again, the initial simulations for model calibration were made using values for the accumulation and washoff parameters as suggested in the user's manual and values for the infiltration and basin-characteristic parameters as determined from the flow calibration.

Comparisons of simulated with measured storm-runoff loads of total nitrogen for the initial simulations are shown in table 6. Simulation errors ranged from 67 to 940 percent. The extent of this overprediction again increases with the value of the model parameter, DRYDAYS. This is consistent with the fact that the data from the Lakewood basin showed no discernible effect of DRYDAYS on storm-runoff loads (fig. 8); whereas, the model assumes that the land-surface load of total nitrogen at the start of a storm is directly proportional to DRYDAYS. The percentage of land-surface load washed off (according to the model) varied from 30 to 78 percent for the three runoff periods of table 6.

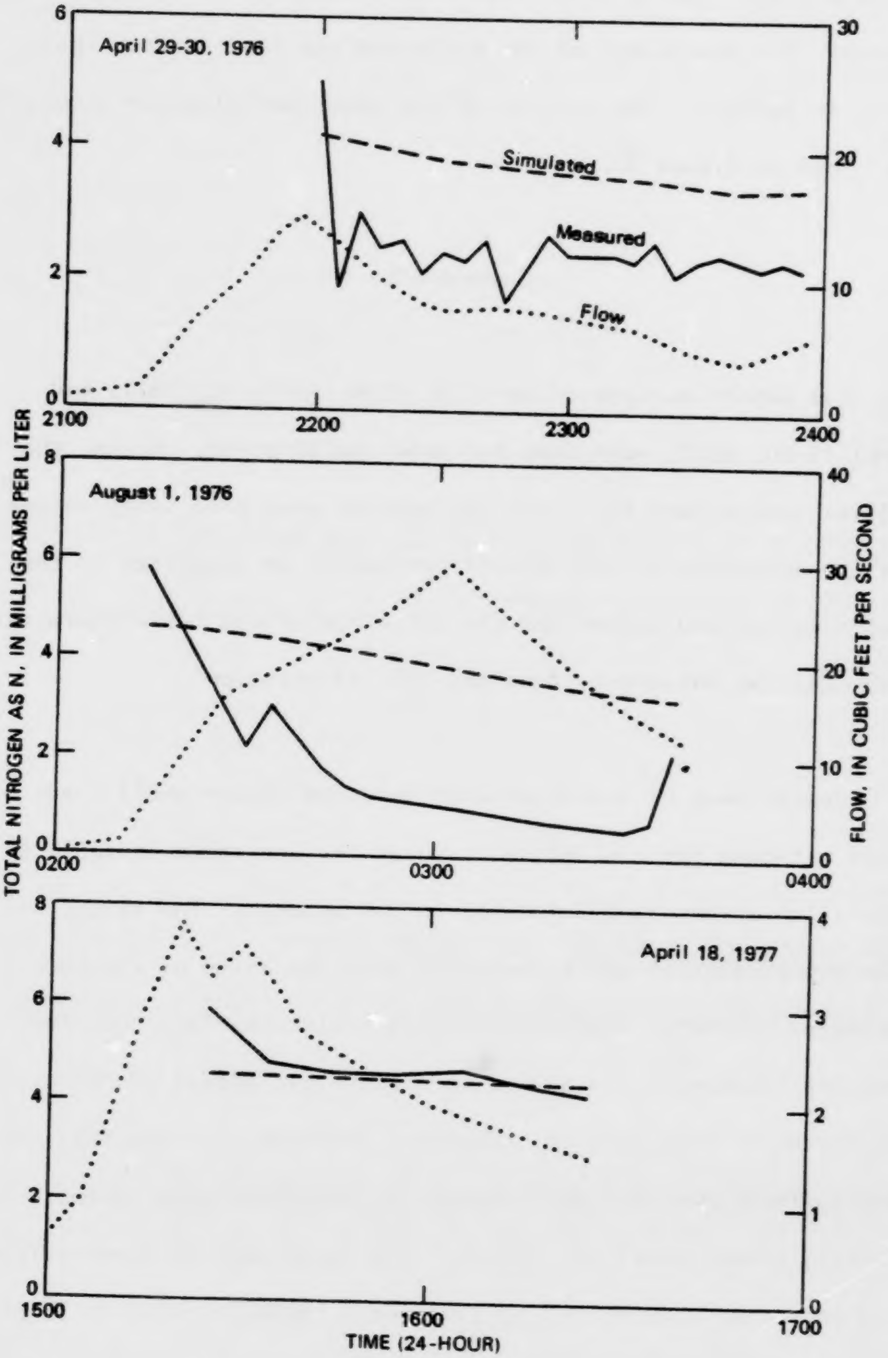

Figure 10.--Simulated and measured changes in total nitrogen with time for runoff periods used in model verification for the Littleton site.

Table 6.--*Measured and simulated storm-runoff loads of total nitrogen from the Lakewood basin*

Date	DRYDAYS[1]	Measured load (pounds)	Simulation 1[2]		Simulation 2[3]	
			Simulated load (pounds)	Percent error	Simulated load (pounds)	Percent error
July 19, 1976------	32	2.6	27	940	1.8	-31
April 12, 1977-----	33	3.7	33	790	2.6	-30
April 19-20, 1977--	7	12	20	67	1.1	-90

[1] DRYDAYS is a model parameter set at the number of days prior to the storm during which the accumulated rainfall is less than 1.0 in.

[2] Model constituent-accumulation and constituent-washoff parameters were set at those values suggested by model documentation (Huber and others, 1975).

[3] Same as simulation 1 except no part of the suspended-solids or settleable-solids concentration was added to the total nitrogen concentration.

A second simulation was made assuming no part of the suspended-solids or settleable-solids concentration was added to the total nitrogen concentration. The results, shown in table 6, again indicate that the model simulations of total nitrogen are very sensitive to these poorly defined model parameters. Again, the large simulation errors suggest that caution should be used when applying the Storm Water Management Model II or other similar models to assist in making water-quality-management decisions, particularly in the absence of local data for model calibration and verification.

The model option to specify the land-surface loads for each subbasin, in pounds per acre, prior to storm runoff was again used in place of the constituent-accumulation equation. The land-surface loads for each of the three runoff periods were adjusted as previously described for the Littleton basin. The results of the "calibrated" model for each runoff period are shown in table 7. No relationship of initial basin-average land-surface load of total nitrogen with antecedent precipitation or number of days since last street sweeping is displayed by these results. The initial basin-average land-surface loads of total nitrogen are similar for the Littleton (table 5) and Lakewood (table 7) sites.

Table 7.--*Basin-average land-surface loads of total nitrogen at the beginning of storm runoff based on model calibration for the Lakewood basin*

Date	Daily rainfall for previous 10 days[1] (inches)										Number of days since last street sweeping	Basin-average land-surface load of total nitrogen as N at the beginning of storm runoff (pound per acre)
	10	9	8	7	6	5	4	3	2	1		
July 19, 1976------	0	0	0	0	0	0	0	0	0	0	46	0.11
Apr. 12, 1977------	0	0	0	0	0	0	0	0	.16	.28	5	.11
Apr. 19-20, 1977---	0	.16	.28	.29	.02	.49	.21	0	.01	.03	12	.21

[1] Refers to number of days prior to date of storm runoff.

The results of the three calibration simulations are shown in figure 11. Three additional runoff periods were sampled for total nitrogen but an insufficient number of samples was collected to provide meaningful storm-runoff loads. However, these runoff periods could be used for model verification of total nitrogen concentrations. Basin-average land-surface load of total nitrogen was assumed to be equal to the average of the basin-average land surface loads shown in table 7. The results of the three verification simulations are shown in figure 12.

SUMMARY

Data from the Littleton and Lakewood basins were used for calibration and verification of the U.S. Environmental Protection Agency's Storm Water Management Model II for rainfall-runoff modeling of flow and total nitrogen. Using values for the accumulation and washoff parameters as suggested in the user's manual (Huber and others, 1975) and basin-characteristic parameters as determined from the flow calibration, errors in simulated storm-runoff loads of total nitrogen ranged from 140 to 1,000 percent at the Littleton site and from 67 to 940 percent at the Lakewood site. The extent of overprediction increased with the value of the model parameter, DRYDAYS. The model assumption that land surface loads are directly proportional to the number of days prior to the storm during which the accumulated rainfall was less than 1.0 in. was not substantiated. The results indicate that the model simulations of total nitrogen are very sensitive to the poorly defined

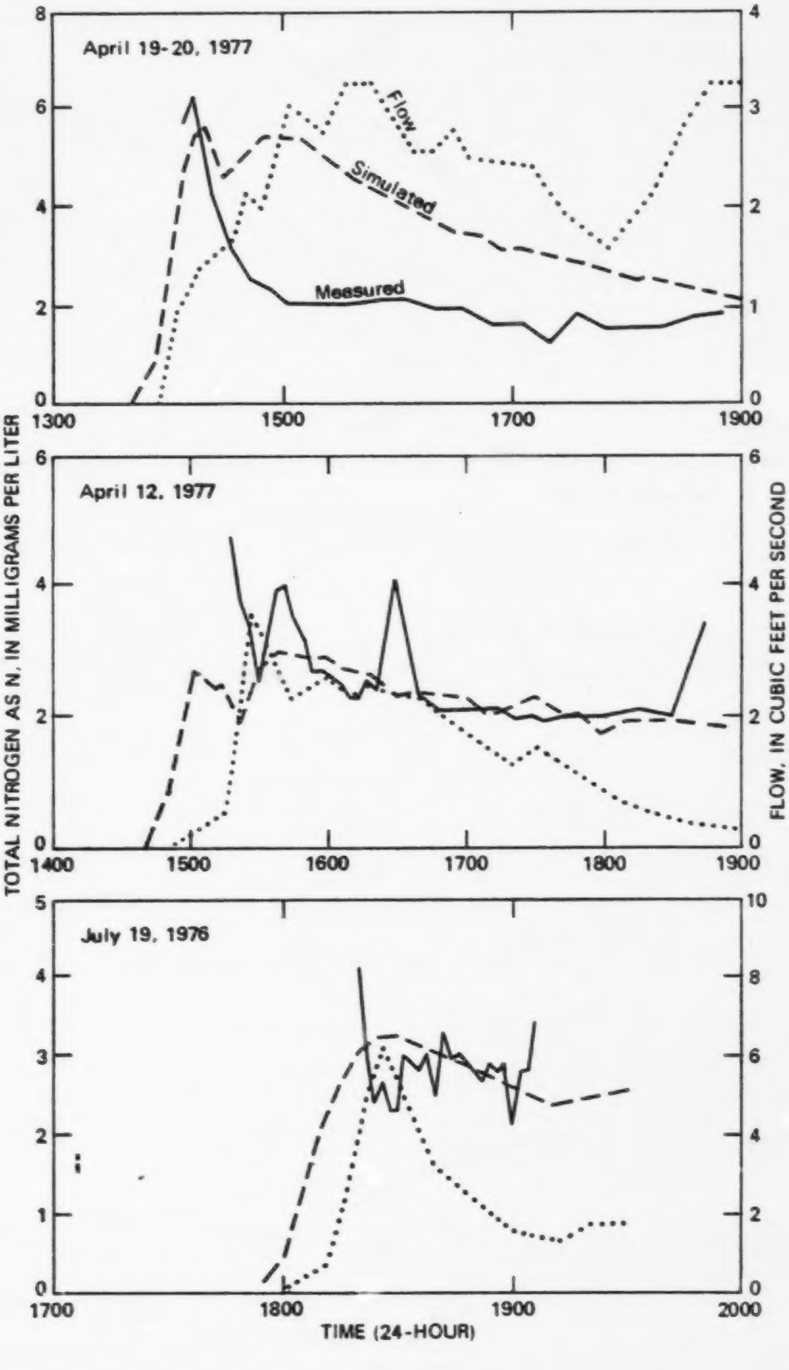

Figure 11.—Simulated and measured changes in total nitrogen with time for runoff periods used in model calibration for the Lakewood site.

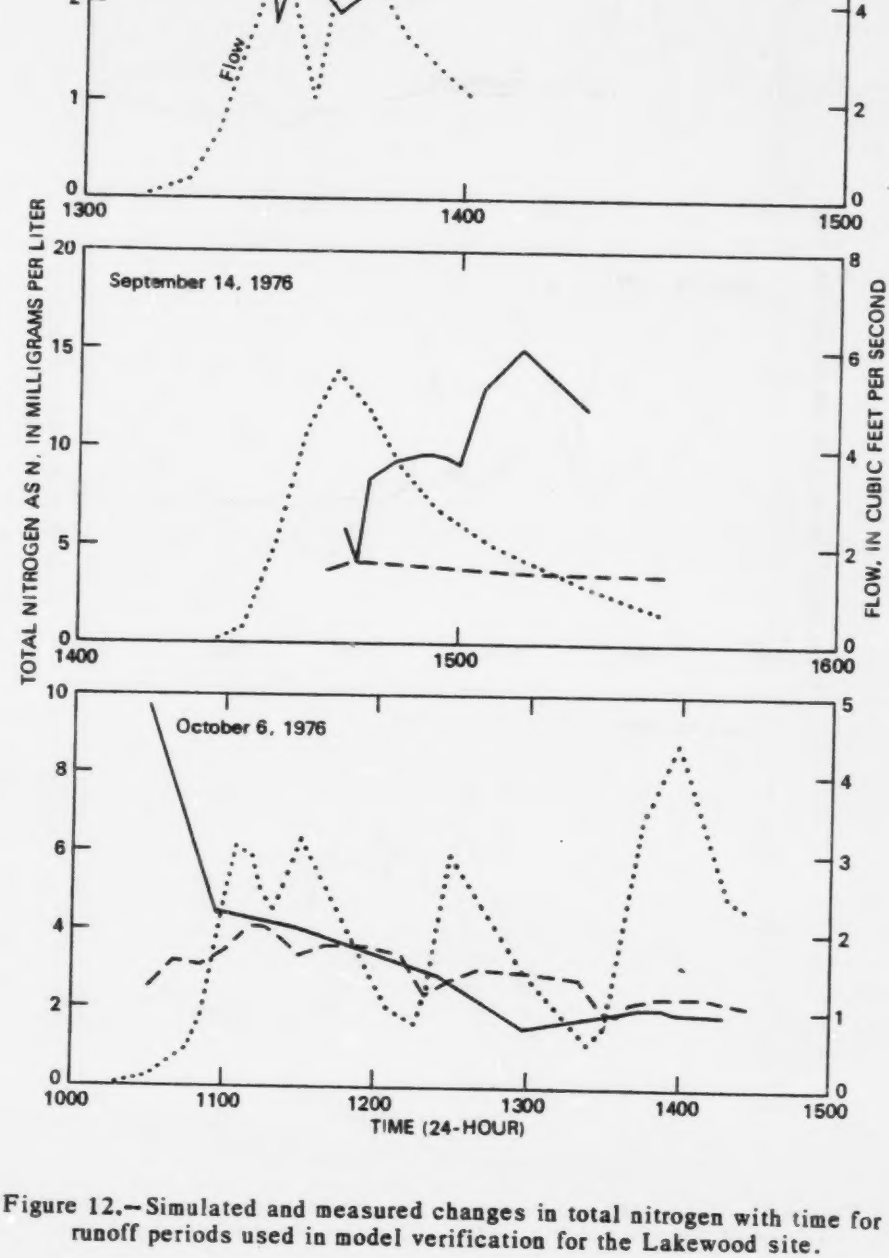

Figure 12.—Simulated and measured changes in total nitrogen with time for runoff periods used in model verification for the Lakewood site.

model parameters specifying the part of the suspended-solids and settleable-solids concentrations added to the total nitrogen concentration. Based on the results of this study, caution should be used when applying the Storm Water Management Model II or other similar models to assist in making water-quality-management decisions, particularly in the absence of local data for model calibration and verification

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Whipple, W., Jr., Hunter, J. V., and Yu, S. L., 1977, Effects of storm frequency on pollution from urban runoff: Water Pollution Control Federation Journal, v. 49, no. 11, p. 2243-2248.

LIST OF ATTENDEES

SWMM USERS GROUP MEETING

MAY 24-25, 1979

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